



Concrete structures



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Australian Standard[®]

Concrete structures

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PREFACE

This Standard was prepared by Standards Australia Committee BD-002, Concrete Structures, to supersede AS 3600—2009.

This Standard incorporates Amendment No. 1 (November 2018) and Amendment No. 2 (May 2021). The changes required by the Amendment are indicated in the text by a marginal bar and amendment number against the clause, note, table, figure or part thereof affected.

The principal objective of this Standard is to provide users with nationally acceptable unified rules for the design and detailing of concrete structures and members, with or without steel reinforcement or prestressing tendons, based on the principles of structural engineering mechanics. The secondary objective is to provide performance criteria against which the finished structure can be assessed for conformance with the relevant design requirements.

The following list indicates the major differences between this edition and the 2009 edition of AS 3600:

- (a) Addition of the following new sections:
 - (i) Section 14 *Design for Earthquakes Actions* (formerly Appendix C).
 - (ii) Section 15 *Diaphragms*.
 - (iii) Section 16 *Steel Fibre Reinforced Concrete*.
 - (iv) Section 18 *Design for Fatigue*.
 - (v) Appendix C *Residual Tensile Strength Test for SFRC*.
- (b) Revision of the following requirements:
 - (i) Phi factors.
 - (ii) Maximum steel strength.
 - (iii) Shear in deep slabs.
 - (iv) Fire design, including—
 - (A) axis distances for fire design;
 - (B) continuous top reinforcement; and
 - (C) minimum slab thickness.
 - (v) Modification of models and calculations of—
 - (A) shrinkage;
 - (B) creep;
 - (C) deflections; and
 - (D) development lengths for higher strength steels.
 - (vi) Steel shrinkage in areas modelled by strut and tie.
 - (vii) Punching shear.
 - (viii) Ductility for pre-cast concrete connections.
 - (ix) Heating and re-bending bars.
 - (x) Crack control.

Statements expressed in mandatory terms in notes to figures and tables are deemed to be requirements of this Standard.

The terms 'normative' and 'informative' are used in Standards to define the application of the appendices to which they apply. A 'normative' appendix is an integral part of a Standard, whereas an 'informative' appendix is only for information and guidance.

CONTENTS

	<i>Page</i>
SECTION 1 SCOPE AND GENERAL	
1.1 SCOPE AND APPLICATION	8
1.2 NORMATIVE REFERENCES	9
1.3 EXISTING STRUCTURES	9
1.4 DOCUMENTATION	9
1.5 CONSTRUCTION	10
1.6 DEFINITIONS	10
1.7 NOTATION	17
SECTION 2 DESIGN PROCEDURES, ACTIONS AND LOADS	
2.1 DESIGN PROCEDURES	34
2.2 DESIGN FOR STRENGTH	35
2.3 DESIGN FOR SERVICEABILITY	39
2.4 DESIGN FOR FATIGUE	41
2.5 ACTIONS AND COMBINATIONS OF ACTIONS	41
SECTION 3 DESIGN PROPERTIES OF MATERIALS	
3.1 PROPERTIES OF CONCRETE	44
3.2 PROPERTIES OF REINFORCEMENT	51
3.3 PROPERTIES OF TENDONS	52
3.4 LOSS OF PRESTRESS IN TENDONS	54
3.5 MATERIAL PROPERTIES FOR NON-LINEAR STRUCTURAL ANALYSIS	57
SECTION 4 DESIGN FOR DURABILITY	
4.1 GENERAL	58
4.2 METHOD OF DESIGN FOR DURABILITY	58
4.3 EXPOSURE CLASSIFICATION	58
4.4 REQUIREMENTS FOR CONCRETE FOR EXPOSURE CLASSIFICATIONS A1, A2, B1, B2, C1 AND C2	61
4.5 REQUIREMENTS FOR CONCRETE FOR EXPOSURE CLASSIFICATION U	62
4.6 ABRASION	62
4.7 FREEZING AND THAWING	62
4.8 AGGRESSIVE SOILS	63
4.9 RESTRICTIONS ON CHEMICAL CONTENT IN CONCRETE	65
4.10 REQUIREMENTS FOR COVER TO REINFORCING STEEL AND TENDONS	65
SECTION 5 DESIGN FOR FIRE RESISTANCE	
5.1 SCOPE	68
5.2 DEFINITIONS	68
5.3 DESIGN PERFORMANCE CRITERIA	70
5.4 FIRE RESISTANCE PERIODS (FRPs) FOR BEAMS	71
5.5 FIRE RESISTANCE PERIODS (FRPs) FOR SLABS	74
5.6 FIRE RESISTANCE PERIODS (FRPs) FOR COLUMNS	77
5.7 FIRE RESISTANCE PERIODS (FRPs) FOR WALLS	81
5.8 INCREASE OF FIRE RESISTANCE PERIODS (FRPs) BY USE OF INSULATING MATERIALS	83

*Page***SECTION 6 METHODS OF STRUCTURAL ANALYSIS**

6.1	GENERAL.....	85
6.2	LINEAR ELASTIC ANALYSIS	88
6.3	ELASTIC ANALYSIS OF FRAMES INCORPORATING SECONDARY BENDING MOMENTS	90
6.4	LINEAR ELASTIC STRESS ANALYSIS.....	90
6.5	NON-LINEAR FRAME ANALYSIS	91
6.6	NON-LINEAR STRESS ANALYSIS.....	91
6.7	PLASTIC METHODS OF ANALYSIS	92
6.8	ANALYSIS USING STRUT-AND-TIE MODELS	93
6.9	IDEALIZED FRAME METHOD OF ANALYSIS	93
6.10	SIMPLIFIED METHODS OF FLEXURAL ANALYSIS	95

SECTION 7 STRUT-AND-TIE MODELLING

7.1	GENERAL.....	103
7.2	CONCRETE STRUTS.....	103
7.3	TIES	108
7.4	NODES.....	108
7.5	ANALYSIS OF STRUT-AND-TIE MODELS	109
7.6	DESIGN BASED ON STRUT-AND-TIE MODELLING.....	109

SECTION 8 DESIGN OF BEAMS FOR STRENGTH AND SERVICEABILITY

8.1	STRENGTH OF BEAMS IN BENDING.....	110
8.2	STRENGTH OF BEAMS IN SHEAR	113
8.3	GENERAL DETAILS FOR BEAMS.....	123
8.4	LONGITUDINAL SHEAR IN COMPOSITE AND MONOLITHIC BEAMS	127
8.5	DEFLECTION OF BEAMS.....	128
8.6	CRACK CONTROL OF BEAMS.....	131
8.7	VIBRATION OF BEAMS	135
8.8	T-BEAMS AND L-BEAMS	135
8.9	SLENDERNESS LIMITS FOR BEAMS	136

SECTION 9 DESIGN OF SLABS FOR STRENGTH AND SERVICEABILITY

9.1	STRENGTH OF SLABS IN BENDING.....	137
9.2	STRUCTURAL INTEGRITY REINFORCEMENT	140
9.3	STRENGTH OF SLABS IN SHEAR.....	141
9.4	DEFLECTION OF SLABS.....	145
9.5	CRACK CONTROL OF SLABS	147
9.6	VIBRATION OF SLABS	151
9.7	MOMENT RESISTING WIDTH FOR ONE-WAY SLABS SUPPORTING CONCENTRATED LOADS.....	151
9.8	LONGITUDINAL SHEAR IN COMPOSITE SLABS.....	151

SECTION 10 DESIGN OF COLUMNS FOR STRENGTH AND SERVICEABILITY

10.1	GENERAL.....	152
10.2	DESIGN PROCEDURES	152
10.3	DESIGN OF SHORT COLUMNS.....	153
10.4	DESIGN OF SLENDER COLUMNS	154
10.5	SLENDERNESS.....	155
10.6	STRENGTH OF COLUMNS IN COMBINED BENDING AND COMPRESSION	159
10.7	REINFORCEMENT REQUIREMENTS FOR COLUMNS.....	162
10.8	TRANSMISSION OF AXIAL FORCE THROUGH FLOOR SYSTEMS.....	170
10.9	CRACK CONTROL	171

SECTION 11 DESIGN OF WALLS

11.1 GENERAL.....	172
11.2 DESIGN PROCEDURES	172
11.3 BRACED WALLS.....	173
11.4 EFFECTIVE HEIGHT.....	173
11.5 SIMPLIFIED DESIGN METHOD FOR WALLS SUBJECT TO VERTICAL COMPRESSION FORCES	174
11.6 DESIGN OF WALLS FOR IN-PLANE SHEAR FORCES	175
11.7 REINFORCEMENT REQUIREMENTS FOR WALLS	176

SECTION 12 DESIGN OF NON-FLEXURAL MEMBERS, END ZONES AND BEARING SURFACES

12.1 GENERAL.....	179
12.2 STRUT-AND-TIE MODELS FOR THE DESIGN OF NON-FLEXURAL MEMBERS.....	179
12.3 ADDITIONAL REQUIREMENTS FOR CONTINUOUS CONCRETE NIBS AND CORBELS	181
12.4 ADDITIONAL REQUIREMENTS FOR STEPPED JOINTS IN BEAMS AND SLABS	181
12.5 ANCHORAGE ZONES FOR PRESTRESSING ANCHORAGES	181
12.6 BEARING SURFACES	183
12.7 CRACK CONTROL	183

SECTION 13 STRESS DEVELOPMENT OF REINFORCEMENT AND TENDONS

13.1 STRESS DEVELOPMENT IN REINFORCEMENT.....	185
13.2 SPLICING OF REINFORCEMENT.....	192
13.3 STRESS DEVELOPMENT IN TENDONS	195
13.4 COUPLING OF TENDONS	199

SECTION 14 DESIGN FOR EARTHQUAKE ACTIONS

14.1 GENERAL.....	197
14.2 DEFINITIONS.....	197
14.3 STRUCTURAL DUCTILITY FACTOR (μ) AND STRUCTURAL PERFORMANCE FACTOR (S_p).....	199
14.4 GENERAL EARTHQUAKE DESIGN REQUIREMENTS	199
14.5 INTERMEDIATE MOMENT-RESISTING FRAMES (IMRFs)	201
14.6 LIMITED DUCTILE STRUCTURAL WALLS	204
14.7 MODERATELY DUCTILE STRUCTURAL WALLS.....	209

SECTION 15 DIAPHRAGMS

15.1 GENERAL.....	210
15.2 DESIGN ACTIONS.....	210
15.3 CAST IN-PLACE TOPPINGS.....	211
15.4 DIAPHRAGM REINFORCEMENT.....	211

SECTION 16 STEEL FIBRE REINFORCED CONCRETE

16.1 GENERAL.....	213
16.2 DEFINITIONS.....	213
16.3 PROPERTIES OF SFRC	214
16.4 DESIGN OF SFRC MEMBERS CONTAINING REINFORCEMENT OR TENDONS	219
16.5 DURABILITY	224
16.6 FIRE	224

	<i>Page</i>
16.7 PRODUCTION OF SFRC	224
SECTION 17 MATERIAL AND CONSTRUCTION REQUIREMENTS	
17.1 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR CONCRETE AND GROUT	228
17.2 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR REINFORCING STEEL	230
17.3 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR PRESTRESSING DUCTS, ANCHORAGES AND TENDONS.....	233
17.4 CONSTRUCTION REQUIREMENTS FOR JOINTS AND EMBEDDED ITEMS	235
17.5 TOLERANCES FOR STRUCTURES AND MEMBERS.....	235
17.6 FORMWORK.....	236
17.7 PREFABRICATED CONCRETE STRUCTURES	239
SECTION 18 DESIGN FOR FATIGUE	
18.1 GENERAL.....	241
18.2 MAXIMUM COMPRESSIVE STRESS IN CONCRETE.....	241
18.3 PLAIN CONCRETE WITH COMPRESSION-TENSION STRESS.....	243
18.4 PLAIN CONCRETE WITH PURE TENSION OR COMBINED TENSION- COMPRESSION STRESS.....	243
18.5 SHEAR LIMITED BY WEB COMPRESSIVE STRESSES	243
18.6 SHEAR IN SLABS	243
18.7 ADJUSTMENT FACTOR FOR BOND BEHAVIOUR IN REINFORCING AND PRESTRESSING STEEL	244
18.8 TENSILE STRESS RANGE IN STEEL	245
18.9 CALCULATION OF STRESSES IN REINFORCEMENT AND TENDONS OF FLEXURAL MEMBERS.....	248
SECTION 19 JOINTS, EMBEDDED ITEMS AND FIXINGS	
19.1 JOINTS.....	249
19.2 EMBEDDED ITEMS.....	250
19.3 FIXINGS	250
SECTION 20 PLAIN CONCRETE PEDESTALS AND FOOTINGS	
20.1 GENERAL.....	252
20.2 DURABILITY	252
20.3 PEDESTALS	252
20.4 FOOTINGS.....	252
SECTION 21 SLAB-ON-GROUND FLOORS, PAVEMENTS AND FOOTINGS	
21.1 GENERAL.....	254
21.2 DESIGN CONSIDERATIONS	254
21.3 FOOTINGS.....	254
APPENDICES	
A REFERENCED DOCUMENTS.....	255
B TESTING OF MEMBERS AND STRUCTURES.....	257
C RESIDUAL TENSILE STRENGTH TEST FOR SFRC	263
BIBLIOGRAPHY.....	263

STANDARDS AUSTRALIA

Australian Standard

Concrete structures

SECTION 1 SCOPE AND GENERAL

1.1 SCOPE AND APPLICATION**1.1.1 Scope**

This Standard sets out minimum requirements for the design and construction of concrete building structures and members that contain reinforcing steel or tendons, or both. It also sets out minimum requirements for plain concrete pedestals and footings.

NOTES:

- 1 The general principles of concrete design and construction and the criteria embodied in this Standard may be appropriate for concrete structures other than buildings, members not specifically mentioned herein and to materials outside the limits given in Clause 1.1.2.
- 2 It is intended that the design of a structure or member to which this Standard applies be carried out by, or under the supervision of, a suitably experienced and competent person.
- 3 For guidance on the design of maritime structures refer to AS 4997.
- 4 If alternate materials and methods to those prescribed in this Standard are to be used, they would need to be considered as part of the development of a Performance Solution to demonstrate compliance with the relevant Performance Requirements of the National Construction Code (NCC) and be accepted by the relevant building authority.

This Standard is not intended to apply to the design of mass concrete structures.

1.1.2 Application

This Standard applies to structures and members in which the materials conform to the following:

(a) Concrete with—

- (i) characteristic cylinder compressive strength at 28 days (f'_c) in the range of 20 MPa to 120 MPa; and
- (ii) with a saturated surface-dry density in the range 1800 kg/m³ to 2800 kg/m³.

(b) Reinforcing steel of Ductility Class N or E in accordance with AS/NZS 4671.

NOTE: These reinforcing materials may be used, without restriction, in all applications referred to in this Standard. This Standard has been written using Ductility Class N reinforcing steels which are readily available in Australia. Where Ductility Class N is referenced in this Standard, the Earthquake Ductility Class E steels may be substituted but the availability of supply in Australia needs to be checked prior to specification on design drawings.

(c) Reinforcing steel of Ductility Class L in accordance with AS/NZS 4671 may be used as main or secondary reinforcement in the form of welded wire mesh, or as wire, bar and mesh in fitments, provided it is not used in any situation where the reinforcement is required to undergo large plastic deformation under strength limit state conditions.

NOTE: The use of Ductility Class L reinforcement is further limited by other clauses within the Standard.

A2

- (d) Higher reinforcing steel grades >500 MPa to 800 MPa meeting the requirements of Table 3.2.1. For ultimate limit states the strength of the reinforcement in design models shall not be taken as greater than 600 MPa unless noted otherwise.
- (e) Prestressing tendons conforming with AS/NZS 4672.1 and tested in accordance with AS/NZS 4672.2.
- (f) Steel fibres conforming to ISO 13270:2013 and meeting the requirements of Clause 16.3.3.8.

1.1.3 Exclusions

The requirements of this Standard shall not take precedence over design requirements and material specifications set out in other Australian Standards that deal with specific types of structures.

1.2 NORMATIVE REFERENCES

Normative documents referred to in this Standard are listed in Appendix A.

NOTE: Informative documents referred to in this Standard are listed in the Bibliography at the end of this document.

1.3 EXISTING STRUCTURES

The general principles of this Standard shall be applied when evaluating the strength or serviceability of an existing structure.

NOTE: Existing structures are likely to contain materials that do not conform with the material specifications herein and may have been designed to different requirements, but the general principles of this Standard would apply. (See also Appendix B.)

1.4 DOCUMENTATION

The drawings and/or specification for concrete structures and members shall include, when required, the following:

- (a) Reference number and date of issue of applicable design Standards.
- (b) Imposed actions (live loads) used in design.
- (c) The earthquake design category determined from AS 1170.4.
- (d) Any constraint on construction assumed in the design.
- (e) Exposure classification for durability.
- (f) Fire resistance level (FRL), if applicable.
- (g) Class and grade designation of concrete.
- (h) Any required properties of the concrete.
- (i) The curing procedure.
- (j) Grade, Ductility Class and type of reinforcement and grade and type of tendons.
- (k) The size, quantity and location of all reinforcement, tendons and structural fixings and the cover to each.
- (l) The location and details of any splices, mechanical connections and welding of any reinforcement or tendon.
- (m) The maximum jacking force to be applied in each tendon and the order in which tendons are to be stressed.
- (n) The shape and size of each member.

- (o) The finish and method of control for unformed surfaces.
- (p) Class of formwork in accordance with AS 3610 and AS 3610.1 as applicable for the surface finish specified.
- (q) The minimum period of time after placing of concrete before stripping of forms and removal of shores.
- (r) The location and details of planned construction and movement joints, and the method to be used for their protection.

1.5 CONSTRUCTION

All concrete structures, designed in accordance with this Standard, shall be constructed so that all the requirements of the design, as contained in the drawings and specifications, are achieved.

1.6 DEFINITIONS

1.6.1 General

For the purposes of this Standard, the definitions below apply.

1.6.2 Administrative definitions

1.6.2.1 *Building authority or other relevant regulatory authority*

The body having statutory powers to control the design and construction of the structure in the area in which the structure is to be constructed.

1.6.2.2 *Drawings*

The diagrams forming part of the documents setting out the work to be executed.

1.6.2.3 *Specification*

The requirements forming part of the documents setting out the work to be executed.

1.6.3 Technical definitions

1.6.3.1 *Action*

Set of concentrated or distributed forces acting on a structure (direct action), or deformation imposed on a structure or constrained within it (indirect action).

NOTE: The term 'load' is also often used to describe direct actions.

1.6.3.2 *Action effects*

Internal forces, bending moments and torsion due to actions (i.e. the stress resultants).

1.6.3.3 *Anchorage zone*

Region between the face of the member where the prestress is applied and the cross-section at which a linear distribution of stress due to prestress is achieved.

1.6.3.4 *Average ambient temperature*

Average value of the daily maximum and minimum ambient temperatures over the defined period at a site.

1.6.3.5 *Average axis distance*

See Clause 5.2.1.

1.6.3.6 *Axis distance*

Distance from the centre-line axis of a longitudinal bar or tendon to the nearest surface exposed to fire (see Figure 5.2.2).

1.6.3.7 *B-region*

Portion of a member in which the assumption that plane sections remain plane can be applied.

1.6.3.8 *Basic creep coefficient*

Mean value of the ratio of final creep strain to elastic strain for a specimen loaded at 28 days under a constant stress of $0.4f'_c$ (see Clause 3.1.8.2).

1.6.3.9 *Bottle-shaped compression field*

Compression field that is wider at mid-length than at its ends [see Figure 7.2.1(c)].

1.6.3.10 *Braced column*

Column in a structure for which the lateral actions, applied at the ends in the direction under consideration, are resisted by components such as masonry infill panels, shear walls or lateral bracing.

1.6.3.11 *Characteristic strength*

Value of the material strength, as assessed by standard test, that is exceeded by 95% of the material (lower characteristic strength).

1.6.3.12 *Closed fitment*

A unit or multiple units of reinforcement used as an external or internal fitment that form a continuous perimeter around a concrete element with the ends of the fitment anchored into the concrete using a minimum of 135° hooks around a longitudinal bar.

1.6.3.13 *Closed tie*

A unit of reinforcement used as an external or internal fitment having a straight leg and minimum of 135° hooks at each end fixed around a longitudinal bar.

1.6.3.14 *Collector*

An element that takes the distributed lateral action from a diaphragm and delivers it to a vertical element. Also known as a drag bar, strut or distributor. A distributor takes the actions from a vertical element and distributes it into the diaphragm.

1.6.3.15 *Column strip*

See Clause 6.1.4.1.

1.6.3.16 *Composite concrete member*

Member consisting of concrete members constructed separately but structurally connected so the member responds as a unit to applied actions.

1.6.3.17 *Concrete*

Mixture of cement, aggregates and water, with or without the addition of chemical admixtures.

1.6.3.18 *Construction joint*

Joint that is located in a structure or part of a structure for convenience of construction and made so that the load-carrying capacity and serviceability of the structure, or part of the structure, will be unimpaired by the inclusion of the joint.

1.6.3.19 *Cover*

Distance between the outside of the reinforcing steel or tendons and the nearest permanent surface of the member, excluding any applied surface finish.

1.6.3.20 Creep coefficient

Mean value of the ratio of creep strain to elastic strain under conditions of constant stress.

1.6.3.21 Critical opening

Opening through the thickness of a slab where an edge, or part of the edge, of the opening is located at a clear distance of less than $2.5b_o$ from the critical shear perimeter [see Figure 9.3(A)(b)].

1.6.3.22 Critical shear perimeter

Perimeter defined by a line geometrically similar to the boundary of the effective area of a support or concentrated load and located at a distance of $d_{om}/2$ therefrom [see Figure 9.3(A)].

1.6.3.23 Cross tie

An internal fitment with a single reinforcement leg anchored at one end with a 135° (minimum) hook and anchored at the other end with a 90° cog. Both the hook and cog are anchored around a longitudinal bar. See Figure 10.7.4.2.

1.6.3.24 D-region

Portion of a member within a distance equal to the member depth (D), from a discontinuity.

1.6.3.25 Design life

Period for which a structure or a structural member is to remain fit for use for its designed purpose with maintenance.

1.6.3.26 Design strip

See Clause 6.1.4.2.

1.6.3.27 Diaphragm

A horizontal or near horizontal concrete element such as a floor or roof used to transmit horizontal actions into the vertical elements of the force resisting system.

1.6.3.28 Direct loading

Loading on a structure that includes the self-weight of its component members and externally applied loads.

1.6.3.29 Discontinuity

Abrupt change in geometry or loading, including prestress.

1.6.3.30 Duct

Conduit (plain or corrugated) to accommodate prestressing tendon(s) for post-tensioned installation.

1.6.3.31 Ductility Class

Designation relating to the ductility of reinforcement ('L' designates 'low', 'N' designates 'normal', 'E' designates 'earthquake').

NOTE: For further information refer to AS/NZS 4671.

1.6.3.32 Durability

Ability of a structure and its component members to perform the functions for which they have been designed, over a specified period of time, when exposed to their environment.

1.6.3.33 Effective area of a support or concentrated load for slabs in shear

Area totally enclosing the actual support or load and for which the perimeter is a minimum [see Figure 9.3(A)].

1.6.3.34 *Effective depth*

Distance from the extreme compressive fibre of the concrete to the resultant tensile force in the reinforcing steel and tendons in that zone, which will be tensile at the ultimate strength condition of pure bending.

1.6.3.35 *Embedded items*

Items, other than reinforcement and tendons, that are embedded in a concrete member or structure.

NOTE: Embedded items include pipes and conduits with their associated fittings, sleeves, permanent inserts for fixings and other purposes, prestressed anchorages, holding-down bolts and other supports.

1.6.3.36 *Exposure classification*

Designation indicative of the most severe environment to which a concrete member is to be subjected during its design life (see Table 4.3).

1.6.3.37 *Fan-shaped compression field*

Compression field that has non-parallel straight sides [see Figure 7.2.1(b)].

1.6.3.38 *Fire resistance*

Ability of a structure or part of it to fulfil its required functions (loadbearing and/or separating function) for a specified fire exposure, for a specified time.

1.6.3.39 *Fire resistance level (FRL)*

Fire resistance periods for structural adequacy, integrity and insulation expressed in that order.

NOTE: Fire resistance levels for structures, parts and elements of construction are specified by the relevant authority [e.g. in the Building Code of Australia (BCA)].

1.6.3.40 *Fire resistance period (FRP)*

Time, in minutes, for a member to reach the specified failure criteria (i.e. structural adequacy, integrity and/or insulation) if tested for fire in accordance with the appropriate Standard.

NOTE: For structures that need to conform with the BCA requirements, the appropriate Standard is AS 1530.4.

1.6.3.41 *Fire-separating function*

Ability of a boundary element of a fire compartment (e.g. wall, floor or roof) to prevent fire spread by passage of flames or hot gases (integrity) or ignition beyond the exposed surface (thermal insulation) during a fire.

NOTE: When tested in accordance with AS 1530.4, prototypes of such members are exposed to fire from only one direction at a time and are assumed to be similarly exposed for the purpose of interpreting Section 5.

1.6.3.42 *Fitment*

Unit of reinforcement commonly used to restrain from buckling the longitudinal reinforcing bars in beams, columns and piles; carry shear, torsion and diagonal tension; act as hangers for longitudinal reinforcement; or provide confinement to the core concrete.

NOTE: Also referred to commonly as a stirrup, ligature or helical reinforcement.

1.6.3.43 *Fixing or fastener or anchor or lifter*

Material cast into or installed into concrete after it has cured for the purpose of maintaining in position reinforcement, tendons, ducts, formwork, or inserts, used to lift concrete members, or to transfer load from other elements into concrete lifting of members.

1.6.3.44 Flat plate

Flat slab without drop panels.

1.6.3.45 Flat slab

Continuous two-way solid or ribbed slab, with or without drop-panels, having at least two spans in each direction, supported internally by columns without beams and supported externally by walls or columns with or without spandrel beams, or both.

1.6.3.46 Footing

Part of a structure in direct contact with and transmitting load to the supporting foundation.

1.6.3.47 Foundation

Soil, subsoil or rock, whether built-up or natural, by which a structure is supported.

1.6.3.48 Grout

A mixture of cement and water, with or without the addition of sand, or chemical admixtures, that is proportioned to produce a material that will subsequently harden to achieve the specified compressive strength.

1.6.3.49 Headed reinforcement

Steel bar that achieves anchorage by means of a head or end plate.

1.6.3.50 Helical reinforcement

Unit of reinforcement that is wound in a helical fashion around the main longitudinal reinforcing bars in a column or pile restraining them from buckling and to carry shear, torsion and diagonal tension or around tendons at an anchorage to resist bursting action effects.

1.6.3.51 Hollow-core slab or wall

Slab or wall having mainly a uniform thickness and containing essentially continuous voids.

1.6.3.52 Initial force

Force immediately after transfer, at a stated position in a tendon.

1.6.3.53 Insulation (fire)

The ability of a fire-separating member, such as a wall or floor, to limit the surface temperature on one side of the member when exposed to fire on the other side.

1.6.3.54 Integrity (fire)

Ability of a fire-separating member to resist the passage of flames or hot gases through the member when exposed to fire on one side.

1.6.3.55 Jacking force

Force in a tendon measured at the jack.

1.6.3.56 Ligature (reinforcement)

See 'Fitment'.

1.6.3.57 Lightweight concrete

Concrete having a saturated surface-dry density in the range of 1800 kg/m³ to 2100 kg/m³.

1.6.3.58 Limit state

Limiting condition at which the structure ceases to fulfil its designed function.

1.6.3.59 *Loadbearing function*

Ability of a structure or member to sustain specified actions during the fire.

1.6.3.60 *Loadbearing wall or column*

Member designed to support or transmit vertical loads additional to its own weight where the design axial force at mid-height of the member is greater than the lesser of $0.03 f'_c A_g$ or $2 A_g$.

1.6.3.61 *Mean strength*

Statistical average of a number of test results representative of the strength of a member, prototype or material.

1.6.3.62 *Middle strip*

See Clause 6.1.4.3.

1.6.3.63 *Movement joint*

Joint that is made between parts of a structure for the specific purpose of permitting relative movement between the parts of the structure on either side of the joint.

1.6.3.64 *Node*

Point in a joint in a strut-and-tie model where the axes of the struts, ties and concentrated forces acting on the joint intersect.

1.6.3.65 *Nodal zone*

Volume of concrete around a node, which is assumed to transfer strut-and-tie forces through the node.

1.6.3.66 *One-way slab*

Slab characterized by flexural action mainly in one direction.

1.6.3.67 *Plain concrete member*

Member either unreinforced, containing less than minimum reinforcement or reinforced but assumed to be unreinforced for the purpose of design.

1.6.3.68 *Post-tensioning*

Tensioning of tendons after the concrete has hardened.

1.6.3.69 *Prestressed concrete*

Concrete into which internal stresses are induced deliberately by tendons.

NOTE: It includes concrete commonly referred to as 'partially prestressed'.

1.6.3.70 *Prestressing steel*

See 'Tendon'.

1.6.3.71 *Pretensioning*

Tensioning of tendons before the concrete is placed.

1.6.3.72 *Prismatic compression field*

Compression field that is parallel sided [see Figure 7.2.1(a)].

1.6.3.73 *Reinforcement*

Steel bar, wire or mesh but not tendons.

NOTE: Commonly referred to as reinforcing steel.

1.6.3.74 Ribbed slab

Slab incorporating parallel ribs in one or two directions.

1.6.3.75 Self-compacting concrete

Concrete that is able to flow and consolidate under its own weight, completely fill the formwork or excavation even in the presence of dense reinforcement, whilst maintaining homogeneity and without the need for additional compaction, and which conforms with specified requirements for slump flow, viscosity and passing ability.

1.6.3.76 Shear wall

Wall that is designed to resist lateral forces acting in or parallel to the plane of the wall.

1.6.3.77 Short column

Column in which the additional bending moments due to slenderness can be taken as zero.

1.6.3.78 Slender column

Column that does not satisfy the requirements for a short column.

1.6.3.79 Span support

See Clause 6.1.4.4.

1.6.3.80 Strength grade

Numerical value of the characteristic compressive strength of concrete at 28 days (f'_c), used in design.

1.6.3.81 Structural adequacy (fire)

Ability of a member to maintain its structural function when exposed to fire.

1.6.3.82 Strut-and-tie model

Truss model made up of struts and ties connected at nodes.

1.6.3.83 Tendon

Wire, strand or bar (or any discrete group of such wires, strands or bars) that is designed to be pretensioned or post-tensioned.

1.6.3.84 Tie

Tension member in a strut-and-tie model.

1.6.3.85 Torsion strip

Strip of slab of width a , whose longitudinal axis is perpendicular to the direction of M_v^* [see Figure 9.3(B)].

1.6.3.86 Transfer

Time of initial transfer of prestressing forces from the tendons to the concrete.

1.6.3.87 Transmission length

Length, at transfer, over which the stress in a pretensioned tendon builds up from zero at one end to its full value.

1.6.3.88 Transverse width

See Clause 6.1.4.5.

1.6.3.89 Two-way slab

Slab characterized by flexural action in two directions.

1.6.3.90 Uniform strain

Strain in the reinforcement at maximum stress, corresponding to the onset of necking.

1.6.3.91 Upper characteristic strength

Value of the material strength, as assessed by standard test, which is exceeded by 5% of the material.

1.7 NOTATION

The symbols used in this Standard, including their definitions, are listed below.

Unless a contrary intention appears, the following applies:

- (a) The symbols used in this Standard shall have the meanings ascribed to them below, with respect to the structure, or member, or condition to which a clause is applied.
- (b) Where non-dimensional ratios are involved, both the numerator and denominator shall be expressed in identical units.
- (c) The dimensional units for length, force and stress, in all expressions or equations, shall be taken as millimetres (mm), newtons (N) and megapascals (MPa) respectively, unless noted otherwise.
- (d) An asterisk (*) placed after a symbol as a superscript (e.g. M_y^*) denotes a design action effect due to the design load.

Symbol	Definition
A_b	= cross-sectional area of a reinforcing bar
$A_{b,fit}$	= cross-sectional area of the fitment
A_{bar}	= the cross-sectional area of the bar (see Clause 13.1.4)
A_c	= smallest cross-sectional area of the concrete strut at any point along its length and measured normal to the line of action of the strut (see Table 5.6.4 and Clause 7.2.3); <i>or</i> = cross-sectional area bounded by the centre-line of the outermost fitments (see Clause 10.7.3.3)
$A_{c,eff}$	= the effective area of concrete in tension surrounding the bars with depth $h_{c,ef}$ equal to the lesser of $2.5(D-d)$, $(D-kd)/3$ or $D/2$ (see Clause 8.6.2.3)
A_{cp}	= total area enclosed by outside perimeter of concrete section (see Clause 8.2.1.2)
A_{ctu}	= area of concrete within the tensile zone (mm^2) at ultimate (see Clause 16.4.2)
A_g	= gross cross-sectional area of a member
A_{head}	= the net bearing area of the head (see Clause 13.1.4)
A_m	= an area enclosed by the median lines of the walls of a single cell (see Clause 8.3.3)
A_{oh}	= area enclosed by centre-line of exterior closed transverse torsion reinforcement, including area of holes (if any) (see Clause 8.2.5.6)
A_p	= cross-sectional area of prestressing steel
A_{pt}	= cross-sectional area of the tendons in the zone that will be tensile under ultimate load conditions

A2

Symbol	Definition
A_s	= cross-sectional area of reinforcement (see Clauses 3.4.3.2 and 13.2.2); <i>or</i> = cross-sectional area of a single anchored bar of diameter d_b (see Clause 13.1.2.3)
A_{sc}	= cross-sectional area of compressive reinforcement
A_{sf}	= area of fully anchored reinforcement crossing the interface
A_{si}	= cross-sectional area of steel bar (tendon, wire) (see Clause 5.2.1) = area of reinforcement in the i th direction crossing a strut (see Clause 7.2.4)
A_{st}	= cross-sectional area of longitudinal tensile reinforcement; <i>or</i> = cross-sectional area of reinforcement in the zone that would be in tension under the design loads if the effects of prestress and axial loads are ignored
A_{sv}	= cross-sectional area of shear reinforcement
$A_{sv.min}$	= cross-sectional area of minimum shear reinforcement
A_{sw}	= cross-sectional area of the bar forming a closed fitment
A_{tr}	= cross-sectional area of a transverse bar along a development or lap length (see Clause 13.1.2.3)
$A_{tr.min}$	= cross-sectional area of the minimum transverse reinforcement along the development length (see Clause 13.1.2.3)
A_o	= a reference area taken as $A_o = 15\,600\text{ mm}^2$ (see Clause 16.4.2) = area enclosed by shear flow path, including any area of holes therein (see Clause 8.2.1.2)
A_1	= a bearing area (see Clause 12.6)
A_2	= largest area of the supporting surface that is geometrically similar to and concentric with A_1 (see Clause 12.6)
a	= a distance; <i>or</i> = shear span, equal to the distance between the centroids of an applied load and a support reaction in a structure (see Clause 7.2.4); <i>or</i> = perpendicular distance from the nearer support to the section under consideration (see Clause 9.6); <i>or</i> = dimension of the critical shear perimeter measured parallel to the direction of M_v^* [see Figure 9.3(B)]
a_i	= axis distance of steel bar (tendon, wire) 'i' from the nearest exposed surface (see Clause 5.2.1)
a_m	= average axis distance (see Clause 5.2.1)
a_s	= axis distance (see Clause 5.2.2)
a_{sup}	= length of a support in the direction of the span (see Clause 6.1.4.4)

Symbol	Definition
b	= width of a rectangular cross-section or member; <i>or</i> = width of beam at the centroid of the bottom reinforcement (see Clause 5.4.1); <i>or</i> = width of ribs [see Table 5.5.2(C) and Table 5.5.2(D)]; <i>or</i>
A2	= smaller column cross-sectional dimension if rectangular, or the column diameter if circular (see Table 5.6.3, Table 5.6.4 and Clause 10.7.4.3)
b_c	= core dimension measured between the centre-lines of the outermost fitments measured across the width of the section (see Clause 10.7.3.3)
b_{ef}	= effective width of a compression face or flange of a member
b_f	= width of the shear plane (see Clause 8.4.3)
b_l	= size of rectangular, or equivalent rectangular column, capital, or bracket, measured in the direction of the span for which moments are being determined (see Clause 14.5.3.2)
b_o	= dimension of an opening (see Clause 9.3.1.2 and 9.3.1.5)
b_t	= size of rectangular, or equivalent rectangular column, capital, or bracket, measured transverse to the direction of the span for which moments are being determined (see Clause 14.5.3.2)
A2	b_v = effective width of a web for shear (see Clause 8.2.1.5)
	b_w = a width of the web; <i>or</i> = minimum thickness of the wall of a hollow section (see Clause 8.3.3)
	c (c_l) = cover to reinforcing steel or tendons
	c_d = a dimension (in millimetres) corresponding to the smaller of the concrete cover to a bar developing stress and half the clear distance to the next parallel bar developing stress, as shown in Figure 13.1.2.2
A2	CMOD = crack mouth opening displacement (see Clause 16.2.1)
	COD = crack opening displacement (see Clause 16.2.2)
	D = overall depth of a cross-section in the plane of bending; <i>or</i> = depth or breadth of the symmetrical prism as appropriate (see Clause 12.5.6)
	D_{Ed} = fatigue damage factor
	D_b = overall depth of a spandrel beam
A2	[Text deleted]
	D_s = overall depth of a slab or drop panel; <i>or</i> = the member depth at the theoretical cut-off point or debonding point (see Clause 8.1.11.1)
	d = effective depth of a cross-section in the plane of bending
	d_b = nominal diameter of a bar, wire or tendon
	d_c = depth of the idealized strut (see Clause 7.2.4); <i>or</i> = core dimension measured between the centre-lines of the outermost fitments measured through the depth of the section (see Clause 10.7.3.3)

	Symbol	Definition
A2	d_d	= diameter of a prestressing duct (see Clause 8.2.6)
	d_f	= diameter of the bar forming the fitment (see Clause 14.5.2.2)
	d_g	= maximum nominal aggregate size (see Clause 8.2.4.2)
	d_i	= mandrel pin diameter for bending reinforcement
	d_o	= distance from the extreme compressive fibre of the concrete to the centroid of the outermost layer of tensile reinforcement or tendons (not less than $0.8D$ for prestressed concrete members)
	d_{om}	= mean value of d_o , averaged around the critical shear perimeter
	d_p	= distance from the extreme compressive fibre of the concrete to the centroid of the tendons in that zone, which will be tensile under ultimate strength conditions
	d_s	= overall dimension measured between centre-lines of the outermost fitments (see Clause 10.7.3.3)
	d_{sc}	= distance from the extreme compressive fibre of the concrete to the centroid of compressive reinforcement (see Clause 8.1.7)
	d_v	= the effective shear depth of the member calculated in accordance with Clause 8.2.1.9
	EC_e	= electrical conductivity (see Clause 4.8.2)
	E_c	= mean value of the modulus of elasticity of concrete at 28 days
	E_{cj}	= mean value of the modulus of elasticity of concrete at the appropriate age, determined in accordance with Clause 3.1.2
	E_d	= design action effect (see Clauses 2.2.2 to 2.2.6)
A2	E_p	= modulus of elasticity of tendons, determined in accordance with Clause 3.3.2
	E_s	= modulus of elasticity of reinforcement, determined in accordance with Clause 3.2.2
	e	= eccentricity of prestressing force or load; <i>or</i> = the base of Napierian logarithms
	e_a	= an additional eccentricity (see Clause 11.5.1)
	F	= total vertical component of the external load carried through the shear span (see Clause 12.2.1)
	F_d	= uniformly distributed design load, factored for strength or serviceability, as appropriate
	$F_{d,ef}$	= effective design service load per unit length or area, used in serviceability design
	F_{Rj}	= load recorded at $CMOD_j$ (see Figure 16.3.3.8)
	$f'_{0.5}$	= characteristic residual tensile strength of concrete (at 28 days) at a Crack Opening Displacement (COD) of 0.5 mm
	$f'_{1.5}$	= characteristic residual tensile strength of a concrete (at 28 days) at a Crack Opening Displacement (COD) of 1.5 mm

Symbol	Definition
$f_{0.5m}$	= mean values of the tensile strength at Crack Opening Displacement (COD) of 0.5 mm
$f_{1.5m}$	= mean values of the tensile strength at Crack Opening Displacement (COD) of 1.5 mm
$f_{c,fat}$	= compressive strength for fatigue effects
f_{cm}	= mean value of cylinder strength (see Clause 3.1.1.2)
f_{cmi}	= mean value of the in situ compressive strength of concrete at the relevant age (see Clause 3.1.1.2 and Table 3.1.2)
f_{ct}	= uniaxial tensile strength of concrete (see Clause 3.1.1.3)
$f_{ct,f}$	= measured flexural tensile strength of concrete (see Clause 3.1.1.3)
$f_{ct,sp}$	= measured splitting tensile strength of concrete (see Clause 3.1.1.3)
f_{cv}	= concrete shear strength (see Clause 9.2.4)
f_{pb}	= characteristic minimum breaking strength (see Clause 3.3.1)
f_{po}	= the stress in prestressing tendons when strain in the surrounding concrete is zero (see Clause 8.2.4.2.1)
f_{py}	= yield strength of tendons determined in accordance with Clause 3.3.1
f_r	= average confining pressure on the core cross-section taken at the level of the fitments (see Clause 10.7.3.3)
$f_{r,eff}$	= effective confining pressure applied to the core of a column (see Clause 10.7.3.3)
f_{si}	= stress in reinforcement in the i th direction crossing a strut
f_{sy}	= characteristic yield strength of reinforcement (referred to as R_e in AS/NZS 4671), determined in accordance with Clause 3.2.1
$f_{sy,f}$	= characteristic yield strength of reinforcement used as fitments
f_{Lm}	= mean flexural tensile strength corresponding to the limit of proportionality, determined in accordance with Clause 16.3.3.8 with $F_{R,j}$ substituted by F_L
$f_{R,1m}$	= mean residual flexural tensile strength corresponding to a CMOD of 0.5 mm, determined in accordance with Clause 16.3.3.7
A2 $f_{R,2m}$	= mean residual flexural tensile strength corresponding to a CMOD of 1.5 mm, determined in accordance with Clause 16.3.3.7
$f_{R,3m}$	= mean residual flexural tensile strength corresponding to a CMOD of 2.5 mm, determined in accordance with Clause 16.3.3.7
A2 $f_{R,4m}$	= mean residual flexural tensile strength corresponding to a CMOD of 3.5 mm, determined in accordance with Clause 16.3.3.7
f'_c	= characteristic compressive (cylinder) strength of concrete at 28 days
f'_{cc}	= characteristic compressive strength of the concrete in the column (see Clause 10.8)
f'_{ce}	= effective compressive strength of the concrete in the joint (see Clause 10.8)
A2 f'_{cp}	= characteristic compressive cylinder strength of concrete at transfer

	Symbol	Definition
	f'_{cs}	= characteristic compressive strength of the concrete in the slab or beams (see Clause 10.8)
	f'_{ct}	= characteristic uniaxial tensile strength of concrete (see Clause 3.1.1.3)
	$f'_{ct,f}$	= characteristic flexural tensile strength of concrete at 28 days (see Clause 3.1.1.3)
A2	f'_w	= characteristic residual tensile strength of SFRC corresponding to a COD w
	G	= permanent action (dead load)
	g	= dead load, per unit length or area
	g_p	= permanent distributed load normal to the shear interface per unit length, newtons per millimetre (N/mm) (see Clause 8.4.3)
A2	H	= overall height of a wall measured from the boundary at its base to the soffit or connection at its top, continuing through any intermediate slabs or walls
	H_w	= floor-to-floor unsupported height of a wall
	H_{we}	= effective height of a wall
	h	= overall depth of a joint (see Clause 10.8)
	h_s	= flange thickness of a ribbed slab
	h_{sp}	= distance between tip of the notch and top of cross section in millimetres (see Clause 16.3.7)
	I	= second moment of area of the uncracked concrete cross-section about the centroidal axis
	I_c	= second moment of area of a column
	I_{cr}	= second moment of area of a cracked section with the reinforcement transformed to an equivalent area of concrete
	I_{ef}	= an effective second moment of area (see Clause 8.5.3 and Clause 16.4.7.3.2)
	$I_{ef,max}$	= maximum effective second moment of area (see Clause 8.5.3)
	I_f	= second moment of area of a flexural member
	J_t	= a torsional modulus
	j	= time after prestressing, in days (see Clause 3.3.4.3)
	K	= a factor that accounts for the weighted average effectiveness of transverse reinforcement in controlling potential splitting cracks along a development or lap splice length (see Clause 13.1.2.3)
	k	= a coefficient, ratio or factor used with and without numerical subscripts
	k_1	= a coefficient that accounts for the bond properties of the bonded reinforcement (see Clause 8.6.2.3)
	k_2	= a coefficient that accounts for the longitudinal strain distribution (see Clause 8.6.2.3)
	k_{3Dt}	= the three-dimensional orientation factor (see Clause C2)

Symbol	Definition
k_{3Dtb}	= three-dimensional orientation factor (see Clause 16.3.3.4)
k_{co}	= cohesion coefficient (see Clause 8.4.3)
k_{cs}	= factor used in serviceability design to take account of the long-term effects of creep and shrinkage
kd	= the depth of the neutral axis on the cracked section (see Clause 8.6.2.3)
k_d	= a factor to reduce the allowable fatigue stress range in reinforcement at the location of a bend radius $d_i/2.k_d = 0.35 + 0.026 (d_i/d_b) \leq 1.0$ (see Clause 18.8)
k_e	= effectiveness factor accounting for the arrangement of the fitments
k_g	= member size factor determined by Equation 16.4.2, where area of concrete within the tensile zone (mm^2) at ultimate is calculated as $A_{ctu} = b_v d_v \cot \theta_v$ (see Clause 16.4.4.2.1)
k_i	= the instantaneous curvature calculated as the slope of the strain diagram in Figure 16.4.7.3.2(b)
k_m	= coefficient calculated in accordance with Clause 10.4.2
k_r	= ratio of the depth, or breadth, of an anchorage bearing plate to the corresponding depth, or breadth, of the symmetrical prism (see Clause 12.5.4)
A2 k_s	= fibre orientation casting bias factor and is taken as 0.64 (see Clause 16.4.4.2.1)
k_u	= neutral axis parameter being the ratio, at ultimate strength under any combination of bending and compression, of the depth to the neutral axis from the extreme compressive fibre to d
k_{uo}	= ratio, at ultimate strength, without axial force of the depth to the neutral axis from the extreme compressive fibre to d_o
L	= centre-to-centre distance between the supports of a flexural member
L_e	= effective length of a column
L_{ef}	= effective span of a member, taken as the lesser of $(L_n + D)$ and L for a beam or slab; <i>or</i> = $L_n + D/2$ for a cantilever
L_l	= distance between centres of lateral restraints or from a lateral restraint to the free edge
L_n	= length of clear span in the direction in which moments are being determined, measured face-to-face of supporting beams, columns or walls, or for a cantilever, the clear projection
L_o	= L minus 0.7 times the sum of the values of a_{sup} at each end of the span (see Clause 6.10.4.2)
L'_o	= smaller value of L_o for adjoining spans (see Clause 6.10.4.5)
L_p	= development length of tendons
L_{pa}	= length of the tendon from the jacking end to a point at a distance ' a ' from that end (see Clause 3.4.2.4)
L_{pt}	= transmission length for pretensioned tendons

Symbol	Definition
L_s	= span between formwork supports (see Clause 17.6.2.4)
L_{sc}	= development length of a bar for a compressive stress less than the yield stress
L_{st}	= development length of a bar for a tensile stress less than the yield stress
$L_{sy,c}$	= development length in compression, being the length of embedment required to develop the yield strength of a deformed bar in compression (see Clause 13.1.5.1)
$L_{sy,cb}$	= basic development length of a deformed bar in compression (see Clause 13.1.5.2)
$L_{sy,t}$	= development length in tension, to develop the characteristic yield strength of a deformed bar in tension (see Clause 13.1.2 and Figure 13.1.2.3)
$L_{sy,t.lap}$	= the tensile lap length for either contact or non-contact splices (see Clause 13.2.2)
$L_{sy,tb}$	= basic development length of a deformed bar in tension (see Clause 13.1.2.2)
L_t	= width of a design strip [see Figure 6.1.4(A)]
L_u	= unsupported length of a column, taken as the clear distance between the faces of members capable of providing lateral support to the column. Where column capitals or haunches are present, L_u is measured to the lowest extremity of the capital or haunch
L_w	= overall length of a wall
L_x	= shorter effective span of a slab supported on four sides
L_y	= longer effective span of a slab supported on four sides
l_b	= length of the bursting zone (see Clause 7.2.4)
l_f	= length of the steel fibre (see Clause C2)
l_x	= shorter span of a two-way slab [see Table 5.5.2(A)]
l_y	= longer span of a two-way slab [see Table 5.5.2(A)]
A2 $l_{o,fi}$	= effective length of a column under fire conditions (see Clause 5.6.3)
M^*	= design bending moment at a cross-section
M_f^*	= design moment in the fire situation (see Table 5.6.4)
M_s^*	= maximum bending moment at the section based on the short-term serviceability load or construction load (see Clause 8.5.3.1)
$M_{s,1}^*$	= design bending moment at the serviceability limit state, calculated with $\psi_s = 1.0$ (see Clauses 8.6.1 and 9.4.1)
M_v^*	= design bending moment to be transferred from a slab to a support
M_x^*, M_y^*	= design bending moment in a column about the major and minor axes respectively; <i>or</i>

Symbol	Definition
	= positive design bending moment, at midspan in a slab, in the x and y direction respectively
M_1^*, M_2^*	= smaller and larger design bending moment respectively at the ends of a column
M_c	= moment used in the calculation of the buckling load (N_c) (see Clause 10.4.4)
$M_{cr,t}$	= bending moment at time t causing cracking of the section with due consideration to prestress, restrained shrinkage and temperature stresses
M_{nb}	= sum of nominal flexural strengths of the beams framing into the joint (see Clause 14.5.6)
M_{nc}	= sum of the nominal flexural strengths of columns framing into the joint (see Clause 14.5.6)
M_o	= total static moment in a span (see Clause 6.10.4.2); <i>or</i>
M_u	= ultimate strength in bending at a cross-section of an eccentrically loaded compressive member
M_{ub}	= particular ultimate strength in bending when $k_{uo} = 0.003/(0.003 + f_{sy}/E_s)$
M_{uo}	= ultimate strength in bending, without axial force, at a cross-section
$(M_{uo})_{min}$	= minimum required strength in bending at a critical cross-section (see Clause 8.1.6.1)
M_{ux}, M_{uy}	= ultimate strength in bending about the major and minor axes respectively of a column under the design axial force N^*
m	= number of fitments legs crossing the confinement plane (see Clause 10.7.3.3)
	= number of intervals with constant amplitude
N	= number of resisting stress cycles, to be calculated from the fatigue strength
N^*	= axial compressive or tensile force on a cross-section
N_{Rsk}	= number of cycles for resistance of constant amplitude stress $\Delta\sigma R_{sk}$
N_f^*	= design axial load in the fire situation (see Clause 5.6.3)
N_c	= buckling load used in column design
N_i	= ultimate number of constant amplitude cycles in interval 'i' that can be carried before failure
N_u	= ultimate strength in compression, or tension, at a cross-section of an eccentrically loaded compression or tension member respectively
	= ultimate strength per unit length of wall (see Clause 11.5.1)
N_{ub}	= particular ultimate strength in compression of a cross-section when $k_{uo} = 0.003/(0.003 + f_{sy}/E_s)$
N_{uo}	= ultimate strength in compression, without bending, of an axially loaded cross-section
N_{uot}	= ultimate strength in tension, without bending, of an axially loaded cross-section

Symbol	Definition
n	= number of bars uniformly spaced around helical reinforcement (see Clause 13.2.4); <i>or</i> = number of laterally restrained longitudinal bars (see Clauses 10.7.3.3 and 10.7.3.4)
n_{bs}	= number of longitudinal bars being developed or spliced at which a potential splitting crack can develop (see Table 13.1.2.3)
n_e	= the effective modular ratio $(1 + \phi_{cc})E_s/E_c$ (see Clause 8.6.2.3)
n_f	= number of fitment bars within longitudinal spacing or pitch s that a potential splitting crack has to cross (see Table 13.1.2.3)
n_i	= foreseen number of constant amplitude cycles in interval 'i'
n_{sc}	= foreseen number of stress cycles during the required design service life (see Clause 18.1)
P	= force in the tendons; <i>or</i> = maximum force occurring at the anchorage during jacking (see Clause 12.5.4); <i>or</i> = applied loads (see Clause 12.2)
P_e	= total effective prestress force allowing for all losses of prestress
P_v	= vertical component of the prestressing force
p	= a reinforcement ratio
p_{cw}	= web reinforcement ratio for compressive reinforcement (see Clause 8.5.3.1)
$p_{p,eff}$	= the reinforcement ratio given by $A_{st}/A_{c,eff}$ (see Clause 8.6.2.3)
p_w	= a reinforcement ratio in a wall; <i>or</i> = web reinforcement ratio for tensile reinforcement (see Clause 8.5.3.1)
Q	= imposed action (live load) including impact, if any
Q_{fat}	= the cyclic load for fatigue design
q	= imposed action (live load) per unit length or area
R	= design relaxation of a tendon, determined in accordance with Clause 3.3.4.3
R_b	= basic relaxation of a tendon, determined in accordance with Clause 3.3.4.2
R_d	= design capacity of a member or structure (equal to ϕR_u or $\phi_{sys} R_{u,sys}$)
R_u	= ultimate strength of a member (see Clause 2.2)
$R_{u,sys}$	= mean capacity of the structure (see Clause 2.2.5)
r	= radius of gyration of a cross-section
$S_{cd,max}$	= maximum compressive stress level
$S_{cd,min}$	= minimum compressive stress level
S_p	= structural performance factor (see Clause 14.3)

Symbol	Definition
s	= centre-to-centre spacing of fitments including shear, torsional or confining reinforcement, measured parallel to the longitudinal axis of a member; <i>or</i> = standard deviation; <i>or</i> = maximum spacing of transverse reinforcement within $L_{sy,c}$, or spacing of fitments, or spacing of successive turns of helical reinforcement, all measured centre-to-centre, in millimetres (see Clause 13.2.4); <i>or</i> = spacing of anchored shear reinforcement crossing interface (see Clause 8.4.3); <i>or</i> = factor related to type of cement
s_b	= clear distance between bars of the non-contact lapped splice (see Figure 13.2.2)
s_m	= spacing of bars being developed, in millimetres
s_{max}	= maximum centre-to-centre spacing of the shear reinforcement (see Clause 8.4.4)
$s_{r,max}$	= the maximum crack spacing (see Clause 8.6.2.3)
T	= a temperature; <i>or</i> = force resultant of transverse tensile stresses (see Clause 12.5.4)
T^*	= torsional moment at a cross-section
T_b^*	= bursting force calculated at the ultimate limit state (see Clause 7.2.4)
$T_{b,s}^*$	= bursting force calculated at the serviceability state (see Clause 7.2.4)
$T_{b,cr}$	= bursting (or splitting) force across a strut caused at the time of cracking of the strut (see Clause 7.2.4)
T_{cd}	= the net tension force caused by flexure shear and axial force on the flexural compressive side of a cross-section (see Clause 8.2.8.2)
T_{cr}	= torsional cracking moment (see Clause 8.2.1.2)
T_{td}	= the net tension force caused by flexure shear and axial force on the flexural tensile side of a cross-section (see Clause 8.2.8.2)
T_u	= ultimate torsional strength
$T_{u,max}$	= ultimate torsional strength of a beam limited by web crushing failure (see Clause 8.3.3)
T_{us}	= torsional resistance (see Clause 8.2.5.6)
T_w	= vertical component of the force carried by the secondary struts (see Clause 12.2)
t	= time
t_d	= difference between the actual effective thickness of the slab and the effective thickness specified in Table 5.5.1, for the required FRP (see Clause 5.8.2)
t_f	= thickness of topping or flange anchored by shear reinforcement (see Clause 8.4.4)

Symbol	Definition
t_h	= hypothetical thickness of a member used in determining creep and shrinkage, taken as $2A_g/u_e$
t_{nom}	= nominal thickness of topping applied (see Clause 5.8.2)
t_w	= thickness of a wall
u	= length of the critical shear perimeter (see Clause 9.3.1.5)
u_c	= the length of the outside perimeter of concrete cross-section (see Clause 8.2.1.2)
u_e	= exposed perimeter of a member cross-section plus half the perimeter of any closed voids contained therein, used to calculate t_h
u_h	= perimeter of the centre-line of the closed transverse torsion reinforcement (see Clause 8.2.1.2)
V^*	= design shear force at a cross-section
V_{eq}^*	= the equivalent factored shear force (see Clause 8.2.1.2)
V_{min}^*	= minimum design shear action calculated for all load combinations (see Clause 2.5.2.2)
V_o	= shear force which would occur at a section when the bending moment at that section was equal to the decompression moment M_o
V_u	= ultimate shear strength
$V_{u,max}$	= ultimate shear strength limited by web crushing failure
V_{uc}	= ultimate shear strength excluding shear reinforcement (see Clause 8.2.3)
V_{uo}	= ultimate shear strength of a slab with no moment transfer (see Clause 9.3.3)
V_{us}	= contribution by shear reinforcement to the ultimate shear strength of a beam or wall (see Clauses 8.2.3 and 11.6.4)
w	= average clear spacing between adjacent tied longitudinal bars (see Clause 10.7.3.3); <i>or</i> = width of loaded area (see Figure 12.2.1) or node [see Figure 7.2.4(A)] = Crack Opening Width (COD) in relation to Steel Fibre Reinforced Concrete (SFRC)
w'_{max}	= characteristic maximum crack width (see Clause 8.6.1) = calculated maximum crack width in a reinforced concrete member at any time after cracking (see Clause 9.5.1)
X	= a dimension [see Figure 9.3(A)]
x	= shorter overall dimension of a rectangular part of a cross-section
Y	= a dimension [see Figure 9.3(A)]
y	= larger overall dimension of a rectangular part of a cross-section
y_1	= larger overall dimension of a closed fitment (see Clause 9.3.1.5)
Z	= section modulus of the uncracked cross-section, referred to the extreme fibre at which flexural cracking occurs (see Clause 8.1.6.1)

Symbol	Definition
z	= projection of the inclined compressive strut normal to the shear span (see Clause 7.2.4); <i>or</i> = internal moment lever arm between the centroids of the flexural compression force and the flexural tension force acting on the section (see Clause 8.2.8.2)
α	= coefficient; <i>or</i> = angle of divergence between bottled shape compression fields and idealized parallel sided strut (see Clause 7.2.4)
α_b	= coefficient for beams (see Clause 8.1.6.1)
α_c	= coefficient (see Clause 10.3.1)
α_n	= coefficient (see Clause 10.6.4)
α_s	= correlation factor (see Clause 10.4.3)
α_{tot}	= sum in radians of the absolute values of successive angular deviations of the prestressing tendon over the length (L_{pa}) (see Clause 3.4.2.4)
α_v	= angle between the inclined shear reinforcement and the longitudinal tensile reinforcement (see Clause 8.2.3.3)
α_x, α_y	= short and long span bending moment coefficients respectively, for slabs supported on four sides (see Clause 6.10.3.2)
β	= an effective compression strength factor (see Clause 2.2.3); <i>or</i> = fixity factor (see Clause 10.5.4); <i>or</i> = a ratio (see Clauses 8.4.2 and 8.5.3.1); <i>or</i> = a factor with or without alphanumeric subscripts (see Clause 8.2.7)
β_{cc}	= factor for strength gain in concrete with time
β_d	= a factor (see Clause 10.4.3)
β_h	= a ratio (see Clause 9.3.1.5)
β_n	= factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone (see Clause 7.4.2)
β_p	= an estimate, in radians per metre (rad/m), of the angular deviation due to wobble effects (see Clause 3.4.2.4)
β_s	= strut efficiency factor (see Clause 7.2.2)
β_x, β_y	= short and long span bending moment coefficients respectively, for slabs supported on four sides (see Clause 6.10.3.2)
γ	= the ratio, under design bending or design combined bending and compression, of the depth of the assumed rectangular compressive stress block to $k_u d$
γ_1, γ_2	= column end restraint coefficients, determined in accordance with Clause 10.5.3
γ_i	= angle between the axis of a strut and the bars in the i th direction of reinforcement crossing that strut (see Clause 7.2.4)
γ_F	= load factor for fatigue load combinations (see Clause 2.5.2.3)
γ_P	= factor for prestress for fatigue load combinations (see Clause 2.5.2.3); <i>or</i>

Symbol	Definition
A2	= load factor for vertical component of prestress (see Clause 8.2)
Δ	= a deflection
$\Delta\sigma_p$	= change in the stress due to the change in length of the prestressed tie (see Clause 7.3.2)
$\Delta\sigma_{Rsk}(n_{sc})$	= resisting stress range relevant to n_{sc} cycles obtained from a characteristic fatigue strength function
$\Delta\sigma_s$	= maximum nominal steel stress range under the acting load combination for steel
$\delta, \delta_b, \delta_s$	= moment magnifiers for slenderness effects (see Clause 10.4)
ε	= a strain
ε_{cc}	= strain due to concrete creep (see Clauses 3.1.8.1 and 3.4.3.3)
ε_{cm}	= the mean strain in the concrete between the cracks (see Clause 8.6.2.3)
ε_{cs}	= design shrinkage strain, determined in accordance with Clause 3.1.7.1
ε_{cs}^*	= final design shrinkage strain of concrete
ε_{csd}	= drying shrinkage strain, determined in accordance with Clause 3.1.7.2
ε_{cse}	= autogenous shrinkage strain, determined in accordance with Clause 3.1.7.2
ε_{cse}^*	= final autogenous shrinkage strain, determined in accordance with Clause 3.1.7.2
$\varepsilon_{csd,b}$	= basic drying shrinkage strain, determined in accordance with Clause 3.1.7.2
$\varepsilon_{csd,b}^*$	= final drying basic shrinkage strain, determined in accordance with Clause 3.1.7.2
ε_{pu}	= strain at maximum stress of a prestressing tendon
ε_{sm}	= the mean strain in the reinforcement at the design loads for the serviceability limit states, including the effects of tension stiffening and any imposed deformations (see Clause 8.6.2.3)
ε_{su}	= uniform strain at maximum stress, corresponding to the onset of necking
ε_x	= longitudinal strain in the concrete at the mid-depth of the section (see Clause 8.2.4.2.2)
η_c	= factor for stress gradient for concrete in the compression zone of a cracked section
η_s	= factor which increases the stress in the reinforcing steel due to differences in bond behaviour between prestressing and reinforcing steel
θ	= angle measured between the axis of the strut and the axis of a tie passing through a common node (see Clauses 7.2.2 and 12.2); <i>or</i> = angle between tie leg and confinement plane (see Clause 10.7.3.3)
θ_v	= angle between the axis of the concrete compression strut and the longitudinal axis of the member (see Clause 8.2.3.3)
λ_1	= a factor being 1.0 for normal concrete and 0.75 for lightweight concrete

Symbol	Definition
λ_{uc}	= a ratio of loads (see Clause 10.4.3)
μ	= friction curvature coefficient (see Clause 3.4.2.4); <i>or</i> = coefficient of friction (see Clause 8.4.3); <i>or</i> = structural ductility factor (see Section 14)
μ_{fi}	= load level for structural adequacy for a column (see Clause 5.6.3)
ν	= Poisson's ratio for concrete, determined in accordance with Clause 3.1.5
ζ	= ratio of bond strength (see Clause 18.7)
ρ	= density of concrete, in kilograms per cubic metre (kg/m ³), determined in accordance with Clause 3.1.3
ρ_p	= transverse compressive pressure, in megapascals, at the ultimate limit state along the development length perpendicular to the plane of splitting (see Clause 13.1.2.3)
ρ_s	= volumetric ratio of the fitments relative to the volume of the core (see Clause 10.7.3.3)
ρ_{wh}	= horizontal reinforcement ratio (see Clause 14.6.7)
ρ_{wv}	= vertical reinforcement ratio (see Clause 14.6.7)
σ	= stress in the context of ultimate and serviceability limit states of steel fibre reinforced concrete (SFRC) (see Clause 16.1)
$\sigma_{c,1}$	= minimum compressive stress within a distance of 300 mm from the surface under the relevant load combination of actions
$\sigma_{c,2}$	= maximum compressive stress within a distance of 300 mm from the surface under the same load combination as that for which $\sigma_{c,1}$ was determined
$\sigma_{c,300}$	= compressive stress at a depth of 300 mm from the extreme compressive fibre for same load case as $\sigma_{c,max}$, but not less than zero (compression is positive) (see Clause 18.2)
$\sigma_{c,max}$	= maximum compressive stress at the extreme fibre under consideration, compression measured positive, $\gamma_F [G, P, Q_{fat}]$
$\sigma_{c,min}$	= minimum compressive stress at the extreme fibre under consideration, taken as zero if tensile, $\gamma_F [\{G, P, \psi_s Q\}, Q_{fat}]$
σ_{ci}	= sustained stress in the concrete at the level of the centroid of the tendons, calculated using the initial prestressing force prior to any time-dependent losses and the sustained portions of all the service loads (see Clause 3.4.3.3)
σ_{cp}	= average intensity of effective prestress in concrete
σ_{cs}	= maximum shrinkage-induced tensile stress on the uncracked section at the extreme fibre at which cracking occurs (see Clause 8.5.3.1)
$\sigma_{ct,min}, \sigma_{ct,max}$	= minimum and maximum tensile stress
$\sigma_{max}, \sigma_{min}$	= the maximum and minimum concrete design compressive stress (see Clause 2.5.2.3)
σ_o	= a constant sustained stress (see Clause 3.1.8.1)

Symbol	Definition
σ_{pa}	= stress in the tendon at a distance 'a', measured from the jacking end (see Clause 3.4.2.4)
$\sigma_{p,ef}$	= effective stress in the tendon after allowing for all losses (see Clause 8.1.8)
σ_{pi}	= stress in the tendon immediately after transfer
σ_{pj}	= stress in the tendon at the jacking end (see Clause 3.4.2.4)
σ_{pu}	= maximum stress that would be reached in a tendon at ultimate strength of a flexural member
σ_s	= maximum nominal steel stress range under the acting load combination (see Clause 18.8)
σ_{sc}	= a compressive stress being developed in a bar in compression (see Clause 13.1.5.4)
σ_{scr}	= tensile steel stress at the serviceability limit state for a beam in flexure or in tension (see Clause 8.6.1) or for a slab in flexure (see Clause 9.4.1)
$\sigma_{scr,1}$	= tensile stress in reinforcement at a cracked section, due to the short-term load combination for the serviceability limit states, calculated with $\psi_s = 1.0$, when direct loads are applied (see Clause 8.6.1)
σ_{st}	= tensile stress in reinforcement (see Clause 13.1.2.4)
$\Sigma(I/L)_c$	= sum of the stiffness in the plane of bending of all the columns meeting at and rigidly connected to the end of the column under consideration (see Clause 10.5.4)
$\Sigma(\beta I/L)_b$	= sum of the stiffness in the plane of bending of all the beams or slabs, or both, meeting at and rigidly connected to the same end of the column under consideration (see Clause 10.5.4)
τ	= age of concrete at the time of loading, in days
τ^*	= design shear stress acting on the interface (see Clause 8.4.2)
τ_u	= unit shear strength (see Clause 8.4.3)
ϕ	= capacity reduction factor for design using linear elastic analysis (see Clause 2.2.2)
ϕ_s	= stress reduction factor for design using linear stress analysis (see Clause 2.2.3)
$\phi_{c,fat}$	= fatigue strength capacity reduction factor for concrete
$\phi_{s,fat}$	= fatigue strength capacity reduction factor for steel
ϕ_{st}	= strength reduction factor for design using strut-and-tie analysis (see Clause 2.2.4)
ϕ_{sys}	= system strength reduction factor for design using non-linear methods of analysis (see Clauses 2.2.5 and 2.2.6)
φ	= sum of the shear associated with development of nominal moment strengths (see Clause 14.5.2.2)
φ_{cc}	= design creep coefficient at any time t (see Clause 3.1.8.3 and Clause 16.4.7.3.3)

Symbol	Definition
ϕ_{cc}^*	= final design creep coefficient (see Clause 3.1.8.3)
$\phi_{cc,b}$	= basic creep coefficient of concrete, determined in accordance with Clause 3.1.8.2
ψ_a	= factor for reduction of imposed (live) floor loads due to area (refer to AS/NZS 1170.1)
ψ_c	= combination imposed action (live load) factor used in assessing the design load for strength (refer to AS/NZS 1170.0)
ψ_s	= short-term imposed action (live load) factor used in assessing the design load for serviceability (refer to AS/NZS 1170.0)
ψ_l	= long-term imposed action (live load) factor used in assessing the design load for serviceability (refer to AS/NZS 1170.0)
Ω	= dimension of node [see Figure 7.2.4(A)]

SECTION 2 DESIGN PROCEDURES, ACTIONS AND LOADS

2.1 DESIGN PROCEDURES

2.1.1 Design for strength and serviceability

Concrete structures shall be designed for ultimate strength and serviceability limit states in accordance with the general principles and procedures for design as set out in AS/NZS 1170.0 and the specific requirements of Clauses 2.2 and 2.3.

Notwithstanding the requirements of Clauses 2.2 and 2.3, it shall be permissible to carry out design checks for strength and serviceability by testing a structure or a component member in accordance with Appendix B.

2.1.2 Design for earthquake actions

Where structures are required by AS 1170.4 to be designed for earthquake actions they shall conform with that Standard, this Standard and the provisions of Section 14 of this Standard. Reinforcement shall be detailed to provide the structure with the assumed ductility.

2.1.3 Design for robustness and structural integrity

Concrete structures shall be designed to be robust in accordance with Section 6 of AS/NZS 1170.0 and BCA Clause BV2, whichever is applicable.

In the detailing of reinforcement and connections for structural integrity, members of a structure shall be effectively tied together to improve integrity of the overall structure. For cast in place concrete, see Sections 8, 9, 10 and 11 for specific requirements. For specific requirements for prefabricated concrete structures, see Section 17.

2.1.4 Design for durability and fire resistance

Concrete structures shall be designed to be—

- (a) durable in accordance with the procedures and criteria given in Section 4; and
- (b) fire resistant in accordance with the procedures and criteria given in Section 5.

2.1.5 Design for fatigue

Fatigue shall be considered in the design of structures and structural elements subject to regular cyclic loads such as vibrating machines, crane-rails, heavy traffic areas, but need not be considered where the foreseen number of stress cycles n_{sc} is less than 10 000.

Design actions for use in fatigue analysis shall be determined in accordance with the methods specified in Clause 6.1.3, except that the moment redistribution for elastic method of Clause 6.2.7 and plastic method of Clause 6.7 are not permitted. Non-linear analysis is permitted; however, sections shall not undergo large plastic deformations under fatigue load combinations.

NOTE: Linear elastic models should generally be used, and reinforced concrete in tension is considered to be cracked. The ratio of moduli of elasticity for steel and concrete should be taken as $E_s/E_c = 10$.

2.1.6 Material properties

The properties of materials used in the design shall be in accordance with Section 3.

When evaluating the behaviour of a concrete structure, member or cross-section, the values of concrete properties used in the calculation shall take into consideration the age of the concrete, rate of loading and expected variations of material properties.

2.2 DESIGN FOR STRENGTH

2.2.1 General

Strength checks for concrete structures and their component members shall be carried out using the procedures specified in Clauses 2.2.2 to 2.2.6, and methods of structural analysis specified in Section 6, as applicable to the strength check procedures being used.

It shall be permissible to use different strength check procedures for different members in a structure, and for the structure as a whole, provided it can be shown that all external actions and forces and calculated internal stress resultants are consistent with the requirements of equilibrium and compatibility for the entire structure.

2.2.2 Strength check procedure for use with linear elastic methods of analysis, with simplified analysis methods and for statically determinate structures

The strength check procedure for use in conjunction with—

- (a) linear elastic methods of analysis of indeterminate structures and members;
- (b) simplified methods of analysis of indeterminate structures and members; and
- (c) static analysis of determinate structures,

shall be carried out as follows:

- (i) It shall be confirmed that the design capacity is equal to or greater than the design action effect, for all critical cross-sections and regions—

$$R_d \geq E_d \quad \dots 2.2.2$$

where

R_d = design capacity (equal to ϕR_u)

E_d = design action effect

- (ii) The design capacity, $R_d = \phi R_u$, shall be obtained using the applicable capacity reduction factor (ϕ), given in Table 2.2.2, and the ultimate strength (R_u), determined in accordance with the relevant sections of this Standard using characteristic values for the material strengths.
- (iii) The design action effect (E_d), shall be determined for the critical combination of factored actions specified in AS/NZS 1170.0 and Clause 2.5 by one of the following methods of analysis:
 - (A) Linear elastic analysis in accordance with Clause 6.2.
 - (B) Linear elastic analysis incorporating secondary bending moments due to lateral joint displacement in accordance with Clause 6.3.
 - (C) One of the simplified methods of analysis in accordance with Clauses 6.9 and 6.10.
 - (D) Equilibrium analysis of a statically determinate structure.

TABLE 2.2.2
CAPACITY REDUCTION FACTORS (ϕ)

Type of action effect	Capacity reduction factor (ϕ)
(a) Axial force without bending:	
(i) Tension	
(A) members with Class N reinforcement and/or tendons	0.85
(B) members with Class L reinforcement	0.65
(ii) Compression	0.65
(b) Bending without axial tension or compression—	
(i) for members with Class N reinforcement and/or tendons	$0.65 \leq (1.24 - 13k_{uo}/12) \leq 0.85$
(ii) for members with Class L reinforcement	0.65
(c) Bending with axial tension—	
(i) for members with Class N reinforcement and/or tendons	$\phi + [(0.85 - \phi) (N_u/N_{uot})]$ and ϕ is obtained from Item (b)(i)
(ii) for members with Class L reinforcement	0.65
(d) Bending with axial compression, where—	
(i) $N_u \geq N_{ub}$	$0.65k_\phi$
(ii) $N_u < N_{ub}$	$0.65k_\phi + [(\phi - 0.65k_\phi) (1 - N_u/N_{ub})]$ and ϕ is obtained from Item (b)
	Short columns with $Q/G \geq 0.25$, $k_\phi = 1.0$, otherwise, $k_\phi = 12/13$
(e) Shear and Torsion	
(i) for members where Class N fitments are provided meeting the requirements of Clause 8.2.1.7, other than for shear strength limited by web crushing	0.75
(ii) otherwise	0.7
(f) Bearing	0.6
(g) Bending, shear and compression in plain concrete	0.6
(h) Bending, shear and tension in fixings	0.6
(i) Bending, shear and axial force in singly reinforced walls forming part of a primary lateral load resisting system	0.65

NOTES:

- 1 In members where Class L reinforcement together with Class N reinforcement and/or tendons are used as longitudinal tensile reinforcement in the design for strength in bending, with or without axial force, the maximum value of ϕ for calculating the member design strength should be taken as 0.65.
- 2 For prestressed beams at transfer $\phi = 0.6$ (see Clause 8.1.6.2).

2.2.3 Strength check procedure for use with linear elastic stress analysis

The strength check procedure for use with a linear elastic stress analysis of a structure or member shall be made as follows:

- (a) The structure or member shall be analysed for the critical combination of factored actions, as specified in AS/NZS 1170.0 and Clause 2.5, by linear stress analysis, in

accordance with Clause 6.4, assuming the concrete to be uncracked, and using accepted principles of mechanics.

- (b) The calculated principal compressive stresses shall not exceed the following value:

$$\phi_s \beta 0.9 f'_c \quad \dots 2.2.3$$

where

ϕ_s = stress reduction factor with values taken from Table 2.2.3

β = an effective compressive strength factor, to be evaluated as follows:

- (i) In regions not containing effective confining reinforcement—

$$\beta = 1.0 \text{ when the principal tensile stress does not exceed } f'_{ct}, \text{ otherwise} \\ \beta = 0.6$$

- (ii) In regions where effective confining reinforcement is provided, β shall be evaluated by rational calculation taking account of the amount of confining steel and the details used, but shall not exceed two.

- (c) Reinforcement and/or tendons shall be provided to carry all of the internal tensile forces, with stresses not exceeding $\phi_s f_{sy}$ and $\phi_s f_{py}$ respectively, where values for the stress reduction factor (ϕ_s) are in accordance with Table 2.2.3.
- (d) In determining the areas of steel reinforcement, it shall be permissible to reduce the peak stresses by averaging the stresses over an area appropriate to the size of the member.
- (e) The stress development of the reinforcement and tendons shall be determined in accordance with Clauses 13.1 and 13.3 respectively.

TABLE 2.2.3
STRESS REDUCTION FACTORS (ϕ_s)

Material	Stress reduction factor (ϕ_s)
Concrete in compression	0.65
Steel in tension	
Class N	0.85
Class L	0.65
Tendons	0.85

2.2.4 Strength check procedure for use with strut-and-tie analysis

The strength check procedure for use with strut-and-tie analysis shall be carried out as follows:

- (a) The strut-and-tie model shall satisfy the requirements of Section 7.
- (b) The forces acting on all struts and ties and nodes shall be determined for the critical combination of factored actions as specified in AS/NZS 1170.0 and Clause 2.4 by an analysis of the strut-and-tie model in accordance with Section 7.
- (c) The compressive force in any concrete strut shall not exceed the design strength of that strut determined in accordance with Clause 7.2.3. The strength reduction factor (ϕ_{st}) to be used in determining the design strength shall be in accordance with Table 2.2.4.
- (d) The tensile force in any tie shall not exceed the design strength of the tie determined in accordance with Clause 7.3.2 where the strength reduction factor (ϕ_{st}) is given in Table 2.2.4.

- (e) The reinforcement and/or tendons in the ties shall be anchored in accordance with Clause 7.3.3.
- (f) The design strength of nodes shall be calculated in accordance with Clause 7.4.2 and shall not be exceeded. The strength reduction factor (ϕ_{st}) shall be in accordance with Table 2.2.4.
- (g) Tie reinforcement shall be provided by Class N reinforcement or tendons.

TABLE 2.2.4

**STRENGTH REDUCTION FACTORS (ϕ_{st})
FOR DESIGN USING STRUT-AND-TIE ANALYSIS**

Material	Strength reduction factor (ϕ_{st})
Concrete in compression	0.65
Steel in tension	0.85
Fibres in tension	0.7

2.2.5 Strength check procedure for use with non-linear analysis of framed structures

The strength check procedure for use with non-linear analysis of framed structures at collapse shall be carried out as follows:

- (a) It shall be confirmed that the design capacity of the structure is equal to or greater than the design action effect—

$$R_d \geq E_d \quad \dots 2.2.5$$

where

R_d = design capacity of the structure

E_d = design action effect

- (b) The design action effect (E_d) is the critical combination of factored actions as specified in AS/NZS 1170.0 and Clause 2.5.
- (c) The design capacity of the structure ($R_d = \phi_{sys} R_{u,sys}$) shall be obtained using the appropriate system strength reduction factor (ϕ_{sys}), given in Table 2.2.5, and the mean capacity of the structure ($R_{u,sys}$) determined for the same combination of actions adopted in Item (b) to evaluate E_d , by using non-linear frame analysis as specified in Clause 6.5, with mean values of material properties.

TABLE 2.2.5

**SYSTEM STRENGTH REDUCTION FACTORS (ϕ_{sys})
(For application with Clauses 2.2.5 and 2.2.6)**

Type of failure	System strength reduction factor (ϕ_{sys})
For structural systems in which the deflections and local deformations at high overload are an order of magnitude greater than those for service conditions; and yielding of the reinforcement and/or the tendon occurs well before the peak load is reached	0.7
In all other cases	0.5 (see Note)

A2

NOTE: Values larger than 0.5 may be used if it can be shown that, at high overload, adequate warning is given of impending collapse.

2.2.6 Strength check procedure for use with non-linear stress analysis

The strength check procedure for use with non-linear stress analysis at collapse shall be carried out as follows:

- (a) It shall be confirmed that the design capacity of the structure or the component member is equal to or greater than the design action effect—

$$R_d \geq E_d \quad \dots 2.2.6$$

where

R_d = design capacity of the structure or component

E_d = design action effect on the structure or the design action effects for a component

- (b) The design action effect (E_d) shall be the critical combination of factored actions (or action effects) as specified in AS/NZS 1170.0 and Clause 2.5.
- (c) The design capacity of the structure (or component) ($R_d = \phi_{\text{sys}} R_{\text{u,sys}}$) shall be obtained using the appropriate system strength reduction factor (ϕ_{sys}) given in Table 2.2.5, and the mean capacity of the structure (or component) ($R_{\text{u,sys}}$) which shall be determined for the same combination of actions adopted for E_d , by non-linear stress analysis as specified in Clause 6.6, with mean values of material properties.

2.3 DESIGN FOR SERVICEABILITY

2.3.1 General

Design checks shall be carried out for all relevant service conditions to ensure the structure will perform to its designed function and purpose.

NOTE: Design limits given or implied in Clauses 2.3.2 and 2.3.3 are based on previous design experience, and reflect requirements for normal structures. In special situations other limits may require further guidance. For further guidance refer to Appendix C of AS/NZS 1170.0.

2.3.2 Deflection

The deflection of beams and slabs under service conditions shall be controlled as follows:

- (a) A limit for the calculated deflection of the member shall be chosen and shall be relevant to the structure and its designed use. The chosen value shall be not greater than the value calculated from the appropriate deflection-to-span ratio given in Table 2.3.2.
- (b) The member shall be designed so that, under the design load for serviceability, the deflections, determined either by calculation or controlled by limiting the span-to-depth ratios in accordance with Clause 8.5 for beams and Clause 9.3 for slabs, do not exceed the deflection limit.

For unbraced frames and multistorey buildings subject to lateral loading, a limit for the inter-storey lateral drift shall be chosen, which does not exceed 1/500 of the storey height. The structure shall be designed so that, under the design lateral load for serviceability, the calculated inter-storey lateral drift does not exceed the chosen value.

TABLE 2.3.2
LIMITS FOR CALCULATED VERTICAL DEFLECTIONS
OF BEAMS AND SLABS

Type of member	Deflection to be considered	Deflection limitation (Δ/L_{ef}) for spans (Notes 1 and 2)	Deflection limitation (Δ/L_{ef}) for cantilevers (Note 4)
All members	The total deflection	1/250	1/125
Members supporting masonry partitions	The deflection that occurs after the addition or attachment of the partitions	1/500 where provision is made to minimize the effect of movement, otherwise 1/1000	1/250 where provision is made to minimize the effect of movement, otherwise 1/500
Members supporting other brittle finishes	The deflection that occurs after the addition or attachment of the finish	Manufacturer's specification but not more than 1/500	Manufacturer's specification but not more than 1/250
Members subjected to vehicular or pedestrian traffic	The imposed action (live load and dynamic impact) deflection	1/800	1/400
Transfer members	Total deflection	1/500 where provision is made to minimize the effect of deflection of the transfer member on the supported structure, otherwise 1/1000	1/250

NOTES:

- 1 In general, deflection limits should be applied to all spanning directions. This includes, but is not limited to, each individual member and the diagonal spans across each design panel. For flat slabs with uniform loadings, only the column strip deflections in each direction need be checked.
- 2 If the location of masonry partitions or other brittle finishes is known and fixed, these deflection limits need only be applied to the length of the member supporting them. Otherwise, the more general requirements of Note 1 should be followed.
- 3 Deflection limits given may not safeguard against ponding.
- 4 For cantilevers, the values of Δ/L_{ef} given in this table apply only if the rotation at the support is included in the calculation of Δ .
- 5 Consideration should be given by the designer to the cumulative effect of deflections, and this should be taken into account when selecting a deflection limit.
- 6 When checking the deflections of transfer members and structures, allowance should be made in the design of the supported members and structure for the deflection of the supporting members. This will normally involve allowance for settling supports and may require continuous bottom reinforcement at settling columns.

2.3.3 Cracking

2.3.3.1 General

Cracking in concrete structures shall be controlled so that structural performance, durability and appearance of the structure are not compromised.

2.3.3.2 Control of cracking

The requirements for cracking set out in Clause 2.3.3.1 shall be deemed to be satisfied by designing the structure and members to conform to the following requirements:

- (a) Flexural cracking in concrete beams and slabs under service conditions shall be controlled in accordance with Clauses 8.6.1, 8.6.2, 8.6.3, 8.6.4, 8.6.5, 9.5.1, 9.5.2, 9.5.4, 9.5.5 or 16.4.7.4, where required.
- (b) Cracking caused by shrinkage and temperature in concrete slabs shall be controlled in accordance with Clause 9.5.3.

- (c) Cracking in concrete walls under service conditions shall be controlled in accordance with Clause 11.7.2.
- (d) Cracking in D-regions under service conditions shall be controlled in accordance with Clause 12.7.
- (e) Cracking at openings and discontinuities and in the vicinity of restraints under service conditions shall be controlled in accordance with Clauses 8.6.1, 9.5.4, or 9.5.5, where required.
- (f) Pre-hardening cracking shall be controlled by specification and construction measures so that the durability, serviceability and/or the behaviour of the structure or member is not adversely affected.

2.3.4 Vibration

Vibration in concrete structures and members shall be controlled so that the serviceability and structural performance are not adversely affected.

2.4 DESIGN FOR FATIGUE

Design for fatigue shall be carried out in accordance with Section 18 using the strength reduction factors in Table 2.4.

TABLE 2.4
STRENGTH REDUCTION FACTORS
FOR FATIGUE

Material	Strength reduction factor
Concrete ($\phi_{c,fat}$)	0.65
Steel ($\phi_{s,fat}$)	0.85

2.5 ACTIONS AND COMBINATIONS OF ACTIONS

2.5.1 Actions and loads

The minimum actions and loads used in the design shall be those set out in AS/NZS 1170.0.

2.5.2 Combinations of actions and loads

2.5.2.1 General

The combinations of actions, loads and forces used in the design shall be in accordance with AS/NZS 1170.0. Additional combinations for prestressed concrete and for fatigue are given in Clauses 2.5.2.2 and 2.5.2.3, respectively.

2.5.2.2 Additional combinations for prestressed members

Except where otherwise stated in the Standard, stress resultants from the effects of prestressing shall be included with a load factor of unity in load combinations for both ultimate and service design.

For the case of permanent action plus prestressing force at transfer, the more severe of the following shall be used:

- (a) $1.15G + 1.15P$; or
- (b) $0.9G + 1.15P$.

NOTE: See also Clauses 6.2.6 and 8.2.1.3.

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A2 | 2.5.2.3 Actions and load combinations for fatigue

The combinations of actions, loads and forces used for fatigue design shall be in accordance with Table 2.5.2.3(A), where ψ_s is the short-term combination factor given in AS 1170.0.

TABLE 2.5.2.3(A)
FATIGUE LOAD COMBINATIONS

The maximum design stress range in the steel $\Delta\sigma$	$E_d = \gamma_F [Q_{fat}]$
The maximum and minimum concrete design compressive stress $\sigma_{max}, \sigma_{min}$	$E_d = \gamma_F [\{G, \gamma_P P, \psi_s Q\}, Q_{fat}]$
The maximum design tensile stress in plain concrete σ_{max}	$E_d = \gamma_F [\{G, \gamma_P P, \psi_s Q\}, Q_{fat}]$

The combinations in the brackets $\{G, \gamma_P P, \psi_s Q\}$ represent the most adverse combination of permanent and non-cyclic service level actions acting with the fatigue design action Q_{fat} . The fatigue design action Q_{fat} to be used shall be the load level determined for the design situation.

A2 | The representative value factor for prestress γ_P for the fatigue load combinations shall be determined in accordance with Table 2.5.2.3(B).

TABLE 2.5.2.3(B)
REPRESENTATIVE VALUE FACTOR FOR PRESTRESS (γ_P)

Tendon description	Increases safety	Decreases safety
Pre-tensioning or unbonded tendons	0.95	1.05
Post-tensioning with bonded tendons	0.90	1.10
When measures are taken (e.g. direct measurements of pre-tensioning)	1.0	1.0

The load factor γ_F for the fatigue load combinations shall be taken as 1.1. If the stress analysis is accurate or conservative, and can be verified by in situ observations, γ_F may be reduced to $\gamma_F = 1.0$.

2.5.3 Construction effects

In determining the critical design conditions for strength and serviceability, account shall be taken of the conditions during construction, and in particular—

- (a) the construction sequence;
- (b) the influence of the schedule for stripping of formwork; and
- (c) the method of back-propping, and its effect on the loads applied during construction.

2.5.4 Arrangements of vertical loads on continuous beams, frames and floor systems

When design checks are carried out for continuous beams and continuous floor systems, for two-dimensional framed structures and for three-dimensional framed structures and floor systems, alternative arrangements of the vertical loads shall be considered in order to determine the critical load combinations. Variations in the load intensity on individual spans shall be considered, including partial loading as specified in AS/NZS 1170.1, together with variations in the loading patterns, whereby some spans are loaded and others unloaded.

The loading arrangements to be considered shall include at least the following:

- (a) The factored permanent action (dead load), without variations in pattern.

- (b) For factored imposed action (live load), where the pattern of loaded and unloaded spans is fixed, the full factored imposed action (live load) applied in the specified pattern.
- (c) For imposed action (live load), where the pattern of loaded and unloaded spans is variable, as follows:
 - (i) For continuous beams and two-dimensional frames and floor systems—
 - (A) the factored imposed action (live load) on alternate spans;
 - (B) the factored imposed action (live load) on any two adjacent spans; and
 - (C) the factored imposed action (live load) on all spans.
 - (ii) For three-dimensional framed structures and floor systems, patterned variations of the factored imposed action (live load) shall be applied in chequerboard arrangements, including the principles of Items (A), (B) and (C) of Item (i) above, to determine the peak design action effects at each critical section.
 - (iii) Notwithstanding Items (i) and (ii), for beams and slabs at the strength limit state, for which the imposed action (live load) (Q) is less than three-quarters of the permanent action (dead load) (G), the factored imposed action (live load) on all spans.

NOTE: The load arrangements listed are the minimum to be considered for design. In particular, for deflection or vibration-sensitive structures and slender floor systems, additional load arrangements need to be considered.

SECTION 3 DESIGN PROPERTIES OF MATERIALS

3.1 PROPERTIES OF CONCRETE

NOTE: Provided that materials properties are determined by testing in accordance with the relevant provisions of AS 1012, characteristic compressive strengths at 28 days of above 100 MPa and up to 120 MPa may be used with the following sections of this Standard: 2 to 8, 10, 13 to 15, 17 and Appendix B.

3.1.1 Strength

3.1.1.1 Characteristic compressive strengths

- (a) The characteristic cylinder compressive strength of concrete at 28 days (f'_c) shall be either—
- (i) taken as equal to the specified strength grade, provided that curing is ensured and that the concrete conforms with AS 1379; or
 - (ii) determined statistically from compressive cylinder strength tests carried out in accordance with AS 1012.9.

The characteristic compressive cylinder strengths of the standard strength grades are 20 MPa, 25 MPa, 32 MPa, 40 MPa, 50 MPa, 65 MPa, 80 MPa, 100 MPa and 120 MPa.

- (b) The characteristic cube compressive strength of grout at 28 days ($f'_{c,cube}$) shall be ascertained in accordance with any of the following methods:
- (i) Taken as equal to the specified strength grade, provided that curing is ensured.
or
 - (ii) Determined statistically from compressive cylinder strength tests carried out in accordance with AS 1012.9 and converted to cube strength using Table 3.1.1.1.
or
 - (iii) Determined statistically from compressive cube strength tests carried out in accordance with AS 1012.9. The cube strength test result shall be modified to an equivalent cylinder strength in accordance with Table 3.1.1.1.

TABLE 3.1.1.1
RELATIONSHIP BETWEEN CYLINDER AND
CUBE STRENGTH AT 28 DAYS

f'_c MPa	20	25	32	40	50	65	80	100	120
$f'_{c,cube}$ MPa	25	32	40	50	60	80	95	120	150

3.1.1.2 Mean in situ compressive strength

In the absence of more accurate data, the mean value of the in situ compressive strength (f_{cmi}) shall be taken as 90% of the mean value of the cylinder strength (f_{cm}) or shall be taken as those given in Table 3.1.2.

3.1.1.3 Tensile strength

The uniaxial tensile strength (f_{ct}) is the maximum stress that concrete can withstand when subjected to uniaxial tension.

The uniaxial tensile strength shall be determined from either the measured flexural tensile strength ($f_{ct,f}$) or from the measured splitting tensile strength ($f_{ct,sp}$) using—

$$f_{ct} = 0.6f_{ct,f} \quad \text{or} \quad f_{ct} = 0.9f_{ct,sp}$$

where $f_{ct,f}$ and $f_{ct,sp}$ are determined statistically from—

- (a) flexural strength tests carried out in accordance with AS 1012.11; or
- (b) indirect tensile strength tests carried out in accordance with AS 1012.10, respectively.

In the absence of more accurate data, the characteristic flexural tensile strength of concrete ($f'_{ct,f}$) and the characteristic uniaxial tensile strength of concrete (f'_{ct}) shall be taken as—

$$f'_{ct,f} = 0.6\sqrt{f'_c} \quad \text{and} \quad f'_{ct} = 0.36\sqrt{f'_c} \quad \text{at 28 days and standard curing,}$$

and where the mean and upper characteristic values are obtained by multiplying these values by 1.4 and 1.8, respectively.

3.1.2 Modulus of elasticity

The mean modulus of elasticity of concrete at the appropriate age (E_{cj}) shall be either—

- (a) taken as equal to—
 - (i) $(\rho^{1.5}) \times (0.043\sqrt{f_{cmi}})$ (in megapascals) when $f_{cmi} \leq 40$ MPa; or
 - (ii) $(\rho^{1.5}) \times (0.024\sqrt{f_{cmi}} + 0.12)$ (in megapascals) when $f_{cmi} > 40$ MPa,
 consideration being given to the fact that this value has a range of $\pm 20\%$;
- (b) determined by test in accordance with AS 1012.17; and
- (c) for standard strength grades at 28 days determined from Table 3.1.2.

TABLE 3.1.2
CONCRETE PROPERTIES AT 28 DAYS

f'_c MPa	20	25	32	40	50	65	80	100	120
f_{cm} MPa	25	31	39	48	59	75	91	110	128
f_{cmi} MPa	22	28	35	43	53	68	82	99	115
E_c MPa	24 000	26 700	30 100	32 800	34 800	37 400	39 600	42 200	44 400

3.1.3 Density

The density of concrete (ρ) shall be determined by test in accordance with either AS 1012.12.1 or AS 1012.12.2.

NOTE: For normal-weight concrete, the density may be taken as 2400 kg/m³.

3.1.4 Stress-strain curves

The stress-strain curve for concrete shall be either—

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- (a) assumed to be of curvilinear form defined by recognized simplified equations; or
- (b) determined from test data.

For design purposes, the shape of the in situ uniaxial compressive stress-strain curve shall be modified so that the maximum stress is $0.9f'_c$.

3.1.5 Poisson's ratio

Poisson's ratio for concrete (ν) shall be either—

- (a) taken as equal to 0.2; or
- (b) determined by test in accordance with AS 1012.17.

3.1.6 Coefficient of thermal expansion

The coefficient of thermal expansion of concrete shall be either—

- (a) taken as equal to $10 \times 10^{-6}/^{\circ}\text{C}$, consideration being given to the fact that this value has a range of $\pm 20\%$; or
- (b) determined from test data.

3.1.7 Shrinkage

3.1.7.1 Calculation of design shrinkage strain

The design shrinkage strain of concrete (ϵ_{cs}) shall be determined—

- (a) from measurements on similar local concrete;
- (b) by tests after eight weeks of drying modified for long-term value, in accordance with AS 1012.13; or
- (c) by calculation in accordance with Clause 3.1.7.2.

3.1.7.2 Design shrinkage strain

When the design shrinkage strain of concrete (ϵ_{cs}) is to be calculated, it shall be determined as the sum of the chemical (autogenous) shrinkage strain (ϵ_{cse}) and the drying shrinkage strain (ϵ_{csd})—

$$\epsilon_{cs} = \epsilon_{cse} + \epsilon_{csd} \quad \dots 3.1.7.2(1)$$

The autogenous shrinkage strain shall be taken as—

$$\epsilon_{cse} = \epsilon_{cse}^* \times (1.0 - e^{-0.07t}) \quad \dots 3.1.7.2(2)$$

where t is the time (in days) after setting and ϵ_{cse}^* is the final autogenous shrinkage strain given by—

$$\text{For } f'_c \leq 50 \text{ MPa } \epsilon_{cse}^* = (0.07f'_c - 0.5) \times 50 \times 10^{-6} \quad \dots 3.1.7.2(3)$$

$$\text{For } f'_c > 50 \text{ MPa } \epsilon_{cse}^* = (0.08f'_c - 1.0) \times 50 \times 10^{-6}$$

At any time t (in days) after the commencement of drying, the drying shrinkage strain shall be taken as—

$$\epsilon_{csd} = k_1 k_4 \epsilon_{csd,b} \quad \dots 3.1.7.2(4)$$

and k_1 is obtained from Figure 3.1.7.2 and k_4 is equal to 0.7 for an arid environment, 0.65 for an interior environment, 0.6 for a temperate inland environment and 0.5 for a tropical or near-coastal or coastal environment.

The basic drying shrinkage strain ($\epsilon_{csd,b}$) is given by—

$$\varepsilon_{\text{csd.b}} = (0.9 - 0.005 f'_c) \times \varepsilon_{\text{csd.b}}^* \quad \dots 3.1.7.2(5)$$

where $\varepsilon_{\text{csd.b}}^* = 800 \times 10^{-6}$ or determined by testing.

NOTE: Concrete exposed to early drying undergoes shrinkage due to capillary suction. This can result in cracking and poor service performance, particularly of exposed slabs. The amount of shrinkage from suction depends on the ambient conditions and the concrete mix, and can exceed the combined shrinkage from other causes. Therefore, it is important to prevent excessive drying of concrete between the commencement of casting and the application of curing at the completion of finishing.

Consideration shall be given to the fact that ε_{cs} has a range of $\pm 30\%$.

Typical final design shrinkage strains after 30 years are found in Table 3.1.7.2.

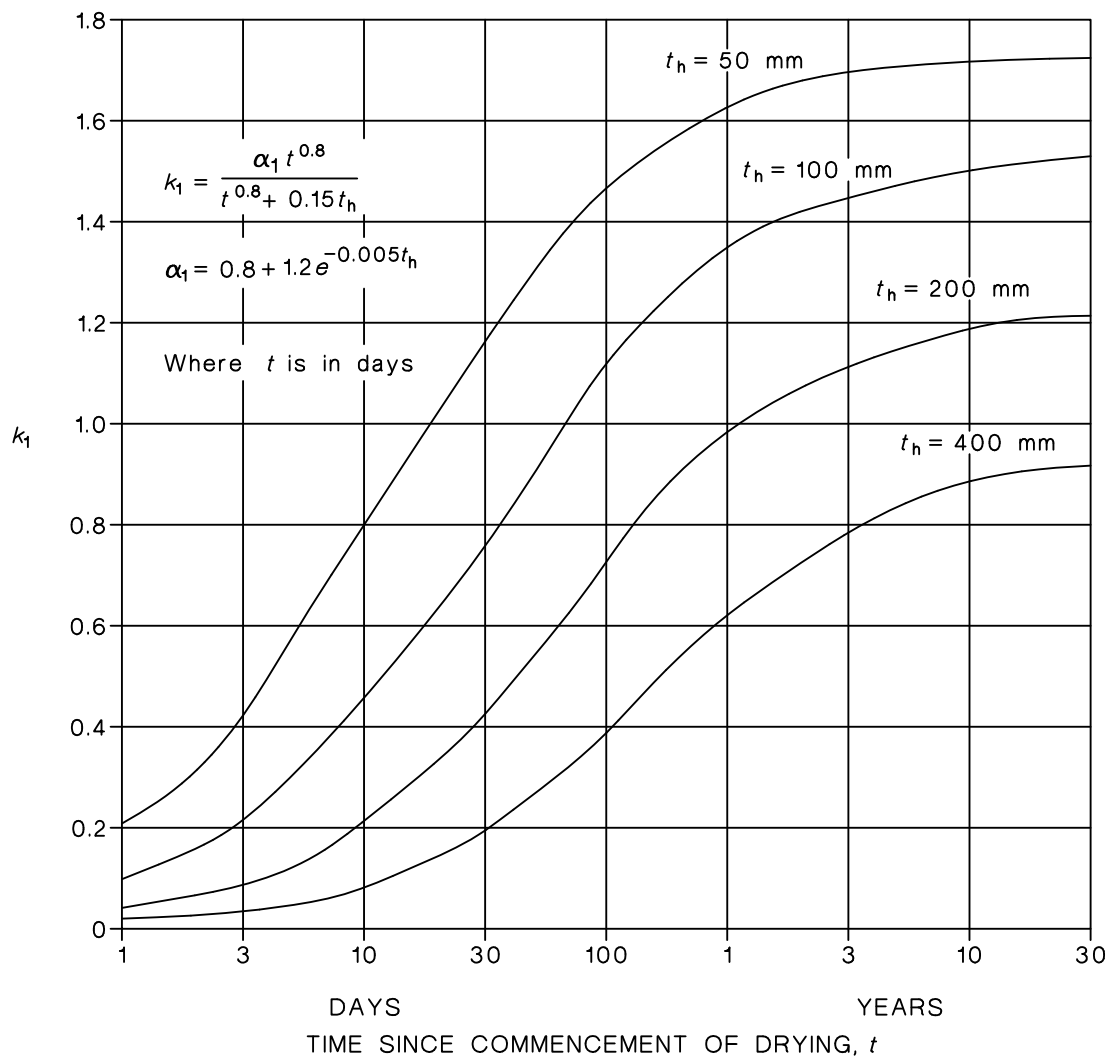


FIGURE 3.1.7.2 SHRINKAGE STRAIN COEFFICIENT (k_1) FOR VARIOUS VALUES OF t_h

TABLE 3.1.7.2
TYPICAL FINAL DESIGN SHRINKAGE STRAINS AFTER 30 YEARS

f'_c (MPa)	Final design shrinkage strain ε_{cs}^* ($\times 10^{-6}$)															
	Arid environment				Interior environment				Temperate inland environment				Tropical, near-coastal and coastal environment			
	t_h (mm)				t_h (mm)				t_h (mm)				t_h (mm)			
	50	100	200	400	50	100	200	400	50	100	200	400	50	100	200	400
25	810	720	590	470	760	670	550	440	710	630	520	410	600	530	440	350
32	800	720	590	470	750	670	560	440	700	630	520	420	600	540	450	360
40	790	710	590	480	740	670	560	450	700	620	530	430	600	540	460	380
50	780	700	590	490	730	660	560	460	690	620	530	440	600	540	470	390
65	770	700	600	510	730	660	570	490	690	630	550	470	610	560	490	420
80	750	690	610	530	720	660	590	510	680	630	560	490	620	570	510	460
100	740	690	620	560	710	660	600	540	680	640	580	530	630	590	540	500

3.1.8 Creep

3.1.8.1 General

The creep strain at any time (t) caused by a constant sustained stress (σ_o) shall be calculated from—

$$\varepsilon_{cc} = \varphi_{cc} \sigma_o / E_c \quad \dots 3.1.8.1$$

where

E_c = mean modulus of elasticity of the concrete at 28 days

φ_{cc} = design creep coefficient at time (t) determined in accordance with Clause 3.1.8.3

3.1.8.2 Basic creep coefficient

The basic creep coefficient of concrete ($\varphi_{cc,b}$) is the mean value of the ratio of final creep strain to elastic strain for a specimen loaded at 28 days under a constant stress of $0.4f'_c$ and shall be—

- determined from measurements on similar local concrete; or
- determined by tests in accordance with AS 1012.16; or
- taken as the value given in Table 3.1.8.2.

TABLE 3.1.8.2
BASIC CREEP COEFFICIENT

Characteristic strength (f'_c), MPa	20	25	32	40	50	65	80	100
Basic creep coefficient ($\varphi_{cc,b}$)	5.2	4.2	3.4	2.8	2.4	2.0	1.7	1.5

3.1.8.3 Design creep coefficient

The design creep coefficient for concrete at any time, t , (φ_{cc}) shall be determined from the basic creep coefficient ($\varphi_{cc,b}$) by any accepted mathematical model for creep behaviour, calibrated such that $\varphi_{cc,b}$ is also predicted by the chosen model.

In the absence of more accurate methods, φ_{cc} at any time shall be taken as—

$$\varphi_{cc} = k_2 k_3 k_4 k_5 k_6 \varphi_{cc,b} \quad \dots 3.1.8.3$$

where k_2 is obtained from Figure 3.1.8.3 and k_3 depends on the age of the concrete (τ) at the time of loading (in days) and is given by the following:

$$k_3 = 2.7/[1 + \log(\tau)] \text{ for } \tau \geq 1 \text{ day}$$

$$k_4 = 0.70 \text{ for an arid environment, } 0.65 \text{ for an interior environment, } 0.60 \text{ for a temperate inland environment and } 0.50 \text{ for a tropical or near-coastal or coastal environment}$$

$$k_5 = \text{a modification factor for high strength concrete, which shall be taken as—}$$

$$k_5 = 1.0 \quad \text{when } f'_c \leq 50 \text{ MPa; or}$$

$$k_5 = (2.0 - \alpha_3) - 0.02(1.0 - \alpha_3) f'_c \quad \text{when } 50 \text{ MPa} < f'_c \leq 100 \text{ MPa}$$

the factor $\alpha_3 = 0.7/(k_4 \alpha_2)$; and α_2 is defined in Figure 3.1.8.3

k_6 accounts for the non-linear creep that develops at sustained stress levels in excess of $0.45f_{cmi}$:

$$k_6 = 1.0 \quad \text{when } \sigma_o \leq 0.45 f_{cmi}$$

$$k_6 = e^{1.5 \left(\frac{\sigma_o}{f_{cmi}} - 0.45 \right)} \quad \text{when } \sigma_o > 0.45 f_{cmi}$$

Consideration shall be given to the fact that φ_{cc} has a range of approximately $\pm 30\%$. This range is likely to be exceeded if the concrete member is subjected to prolonged periods of temperature in excess of 25°C .

The final design creep coefficients (φ_{cc}^*) (after 30 years) predicted by this method for concrete first loaded at 28 days with a sustained stress level not exceeding $0.45f_{cmi}$ are given in Table 3.1.8.3.

NOTE: The compressive stress in concrete caused by the permanent effects (including prestress) should not exceed $0.45f_{cmi}$.

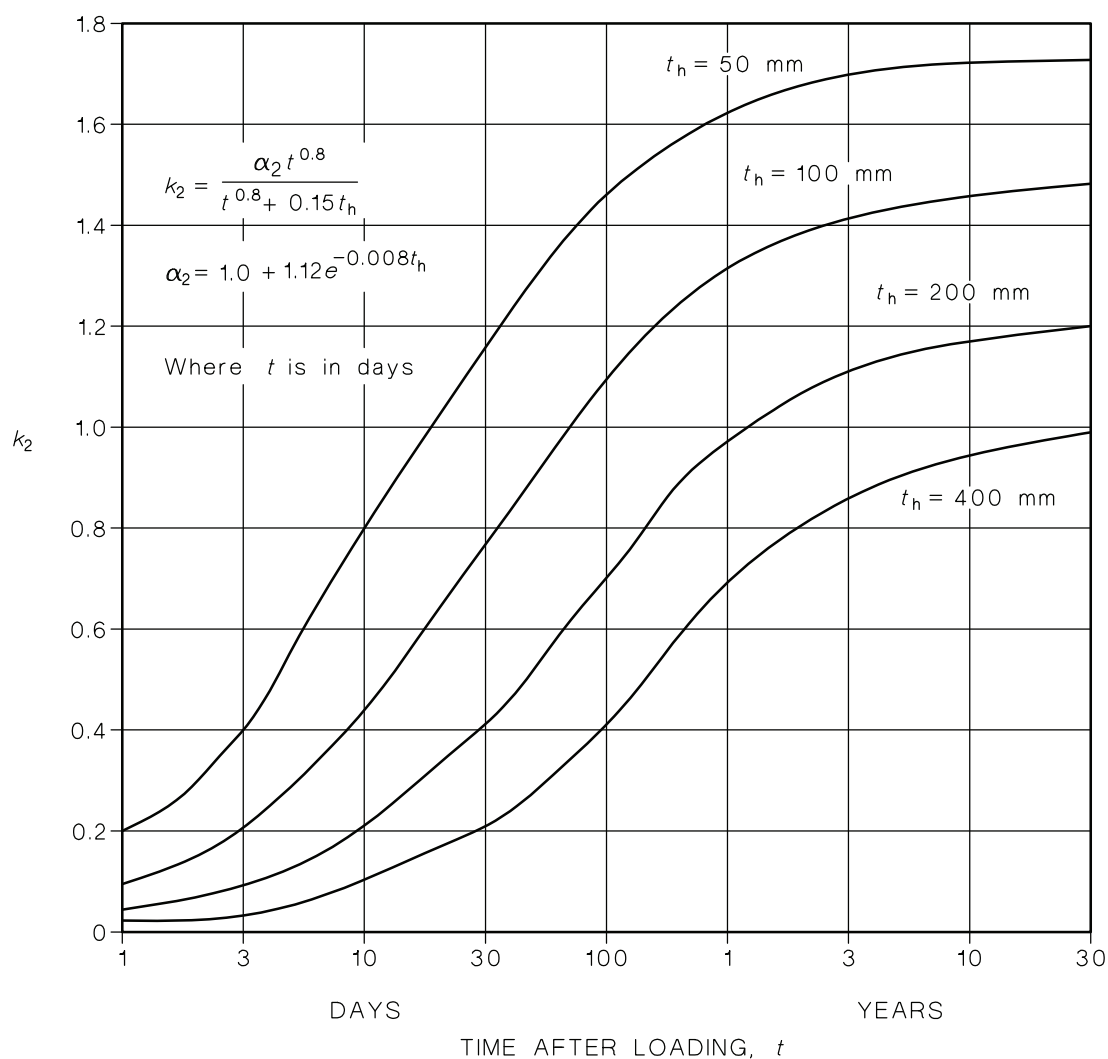
FIGURE 3.1.8.3 COEFFICIENT (k_2)

TABLE 3.1.8.3
FINAL CREEP COEFFICIENTS (AFTER 30 YEARS)
FOR CONCRETE FIRST LOADED AT 28 DAYS

f'_c (MPa)	Final creep coefficient (ϕ_{cc}^*)											
	Arid environment			Interior environment			Temperate inland environment			Tropical, near-coastal and coastal environment		
	t_h (mm)			t_h (mm)			t_h (mm)			t_h (mm)		
	100	200	400	100	200	400	100	200	400	100	200	400
25	4.82	3.90	3.27	4.48	3.62	3.03	4.13	3.34	2.80	3.44	2.78	2.33
32	3.90	3.15	2.64	3.62	2.93	2.46	3.34	2.70	2.27	2.79	2.25	1.90
40	3.21	2.60	2.18	2.98	2.41	2.02	2.75	2.23	1.87	2.30	1.86	1.56
50	2.75	2.23	1.89	2.56	2.07	1.73	2.36	1.91	1.60	1.97	1.59	1.33
65	2.07	1.75	1.53	1.95	1.66	1.46	1.84	1.59	1.38	1.61	1.38	1.23
80	1.56	1.40	1.29	1.50	1.36	1.25	1.45	1.32	1.22	1.33	1.23	1.14
100	1.15	1.14	1.11	1.15	1.14	1.11	1.15	1.14	1.11	1.15	1.14	1.11

3.2 PROPERTIES OF REINFORCEMENT

3.2.1 Strength and ductility

For the purposes of design, the characteristic yield strength of reinforcement (f_{sy}) shall be taken as not greater than the value specified in Table 3.2.1 for the type of reinforcement (see also Clause 17.2.1.1).

The ductility of the reinforcement shall be characterized by its uniform strain (ϵ_{su}) and tensile-to-yield stress ratio and designated as low (L) or normal (N) Ductility Class as given in Table 3.2.1. For the purposes of design, values of these parameters for each Ductility Class shall conform with AS/NZS 4671.

NOTE: In AS/NZS 4671, ϵ_{su} is referred to as A_{gt} , expressed as a percentage, and f_{sy} is referred to as R_e .

TABLE 3.2.1
YIELD STRENGTH AND DUCTILITY CLASS OF REINFORCEMENT

Reinforcement		Characteristic yield strength (f_{sy}) MPa	Characteristic uniform strain (ϵ_{su})	Ductility Class
Type	Designation grade			
Bar plain to AS/NZS 4671	R250N	250	0.05	N
Bar deformed to AS/NZS 4671	D500L (fitments only)	500	0.015	L
	D500N	500	0.05	N
Welded mesh, plain, deformed or indented to AS/NZS 4671	D500L	500	0.015	L
	D500N	500	0.05	N
Stainless steel plain bar to BS 6744	200	200	0.05	N or E
Stainless steel ribbed bar to BS 6744	500	500	0.05	N or E

NOTES:

- 1 Reference should be made to AS/NZS 4671 for explanation to designations applying to 500 MPa steels.
- 2 For higher reinforcing steel grades permitted in Clause 1.1.2(d) the following characteristic properties shall be met:
 - (a) The following limits for the chemical composition determined by cast analysis shall not be exceeded:
 - (i) Carbon – 0.33%, Phosphorus – 0.050%, Sulphur – 0.050%.
 - (ii) The carbon equivalent value shall not exceed 0.49.
 - (b) The maximum yield strength does not exceed the nominal yield strength by more than 150 MPa.
 - (c) For steels:
 - (i) 500 MPa < f_{sy} ≤ 700 MPa: uniform elongation $\epsilon_{su} \geq 0.05$ and the tensile-to-yield stress ratio $R_m/R_e \geq 1.08$.
 - (ii) 700 MPa < f_{sy} ≤ 800 MPa: uniform elongation $\epsilon_{su} \geq 0.04$ and the tensile-to-yield stress ratio $R_m/R_e \geq 1.04$.

3.2.2 Modulus of elasticity

The modulus of elasticity of reinforcement (E_s) for all stress values not greater than the yield strength (f_{sy}) shall be either—

- (a) taken as equal to 200×10^3 MPa; or
- (b) determined by test.

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3.2.3 Stress-strain curves

A stress-strain curve for reinforcement shall be either—

- (a) assumed to be of a form defined by recognized simplified equations; or
- (b) determined from test data.

3.2.4 Coefficient of thermal expansion

The coefficient of thermal expansion of reinforcement shall be either—

- (a) taken as equal to $12 \times 10^{-6}/^{\circ}\text{C}$; or
- (b) determined from test data.

3.3 PROPERTIES OF TENDONS

3.3.1 Strength

The following applies:

- (a) The characteristic minimum breaking strength (f_{pb}) for commonly used tendons shall be as specified in Table 3.3.1. For tendons of dimensions not covered in Clause 3.3, refer to AS/NZS 4672.1.
- (b) The yield strength of tendons (f_{py}) shall be taken either as the 0.1% proof stress as specified in AS/NZS 4672.1, or determined by test data. In the absence of test data it shall be taken as follows:
 - (i) For wire used in the as-drawn condition $0.80f_{pb}$.
 - (ii) For stress-relieved wire $0.83f_{pb}$.
 - (iii) For all grades of strand $0.82f_{pb}$.
 - (iv) For hot-rolled bars (super grade) $0.81f_{pb}$.
 - (v) For hot-rolled ribbed bars $0.89f_{pb}$.

TABLE 3.3.1
TENSILE STRENGTH OF COMMONLY USED WIRE STRAND AND BAR

Material type and Standard	Nominal diameter mm	Area mm ²	Characteristic minimum breaking load kN	Characteristic minimum breaking strength (f_{pb}) MPa
As-drawn wire, AS/NZS 4672.1	5.0	19.6	34.7	1700
	7.0	38.5	64.3	1670
Stress-relieved wire, AS/NZS 4672.1	5.0	19.9	33.8	1700
	7.0	38.5	64.3	1670
7 wire ordinary strand, AS/NZS 4672.1	9.5	55.0	102	1850
	12.7	98.6	184	1870
	15.2	143	250	1790
	15.2	143	261	1830
	15.7	150	279	1860
7 wire compacted strand, AS/NZS 4672.1	15.2	165	300	1820
	18.0	223	380	1700

Hot-rolled bars, AS/NZS 4672.1 (Super grade only)	26	562	579	1030
	29	693	714	1030
	32	840	865	1030
	36	995	1025	1030
	40	1232	1269	1030
	56	2428	2501	1030
	75	4371	4502	1030

3.3.2 Modulus of elasticity

The modulus of elasticity of commonly used tendons (E_p) shall be either—

- (a) taken as equal to—
 - (i) for as-drawn wire, to AS/NZS 4672.1 $205 \pm 10 \times 10^3$ MPa;
 - (ii) for stress-relieved steel wire, to AS/NZS 4672.1 $205 \pm 10 \times 10^3$ MPa;
 - (iii) for stress-relieved steel strand, to AS/NZS 4672.1 $200 \pm 5 \times 10^3$ MPa; or
 - (iv) for hot-rolled high tensile alloy steel bars, to AS/NZS 4672.1 $205 \pm 10 \times 10^3$ MPa; or
- (b) determined by test.

NOTE: Consideration should be given to the fact that the modulus of elasticity of tendons may vary by $\pm 10\%$ and will vary more when a multi-strand or multi-wire tendon is stressed as a single cable. This will influence the calculated extension.

3.3.3 Stress-strain curves

A stress-strain curve for tendons shall be determined from test data.

3.3.4 Relaxation of tendons

3.3.4.1 General

This clause applies to the relaxation, at any age and stress level, of low-relaxation wire, low-relaxation strand, and alloy-steel bars.

3.3.4.2 Basic relaxation

The basic relaxation of a tendon (R_b) after one thousand hours at 20°C with an initial force of $0.8f_{pb}$ for wire and strand and $0.7f_{pb}$ for hot rolled steel bars shall be determined in accordance with AS/NZS 4672.1.

[Text deleted]

3.3.4.3 Design relaxation

The design relaxation of a tendon (R) shall be determined from—

$$R = k_7 k_8 k_9 R_b \quad \dots 3.3.4.3$$

where

k_7 = a coefficient dependent on the duration of the prestressing force

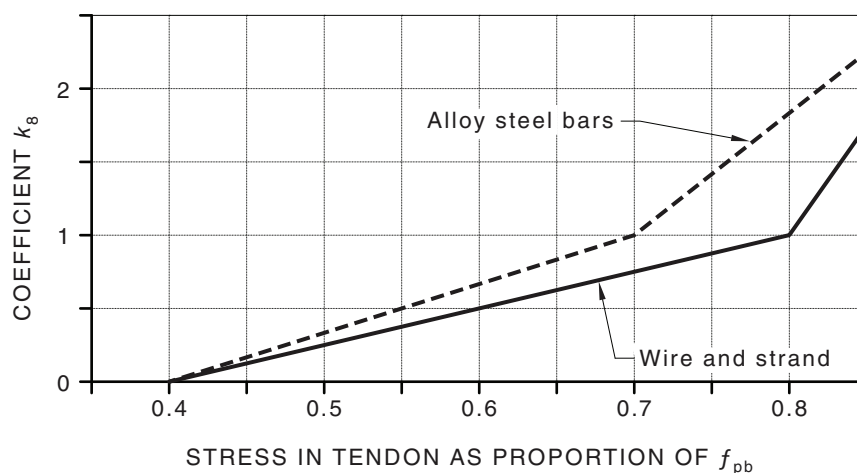
$$= \log [5.4(j)^{1/6}]$$

j = time after prestressing, in days

k_8 = a coefficient, dependent on the stress in the tendon as a proportion of f_{pb} , determined from Figure 3.3.4.3

k_9 = a function, dependent on the average annual temperature (T) in degrees Celsius, taken as $T/20$ but not less than 1.0

When determining the design relaxation, consideration shall be given to the effects of curing at elevated temperatures, if applicable.

FIGURE 3.3.4.3 COEFFICIENT k_8

3.4 LOSS OF PRESTRESS IN TENDONS

3.4.1 General

The loss of prestress in tendons, at any given time, shall be taken to be the sum of the immediate loss of prestress and the time-dependent loss of prestress, calculated in accordance with Clauses 3.4.2 and 3.4.3 respectively.

For structures designed to operate above 40°C, special calculations, based on test data, shall be made.

NOTE: Tendons in structural members subject to elevated temperatures permanently or in a high temperature environment will have significantly higher losses of prestress. Reference to specialist literature is recommended to establish the effect of elevated temperatures on the behaviour of the materials.

3.4.2 Immediate loss of prestress

3.4.2.1 General

The immediate loss of prestress shall be estimated by adding the calculated losses of prestress due to elastic deformation of concrete, friction, anchoring and other immediate losses as are applicable.

3.4.2.2 Loss of prestress due to curing conditions

Where curing of a prestressed member is carried out at ambient conditions, the design relaxation shall be as determined by Clause 3.3.4.3.

Where curing of a prestressed member is carried out at elevated temperature (such as steam curing), a part or all of the design relaxation, as determined from Clause 3.3.4.3, shall be deemed to be part of the immediate loss of prestress.

3.4.2.3 Loss of prestress due to elastic deformation of concrete

Calculation of the immediate loss of prestress due to elastic deformation of the concrete at transfer shall be based on the value of modulus of elasticity of the concrete at that age.

3.4.2.4 Loss of prestress due to friction

The stress variation along the design profile of a tendon due to friction in the jack, the anchorage and the duct shall be assessed in order to obtain an estimate of the prestressing forces at the critical sections considered in the design.

The extension of the tendon shall be calculated allowing for the variation in tension along its length, as follows:

- (a) *Friction in the jack and anchorage* The loss of prestress due to friction in the jack and anchorage shall be determined for the type of jack and anchorage system to be used.
- (b) *Friction along the tendon* Friction loss shall be calculated from an analysis of the forces exerted by the tendon on the duct. In the absence of more detailed calculations the stress in the tendon at a distance (a) measured from the jacking end (σ_{pa}) shall be taken as—

$$\sigma_{pa} = \sigma_{pj} e^{-\mu(\alpha_{tot} + \beta_p L_{pa})} \quad \dots 3.4.2.4$$

where

σ_{pj} = stress in the tendon at the jacking end

e = base of Napierian logarithms

μ = friction curvature coefficient for different conditions, which, in the absence of specific data and when all tendons in contact in the one duct are stressed simultaneously, shall be taken as—

- (i) for greased-and-wrapped coating, 0.15;
- (ii) for bright and zinc-coated metal sheathing, 0.15 to 0.20;
- (iii) for bright and zinc-coated flat metal ducts, 0.20.

α_{tot} = sum in radians of the absolute values of successive angular deviations of the prestressing tendon over the length of the tendon from the jacking end to a point at distance (a) from that end (L_{pa})

β_p = an estimate, in radians per metre (rad/m), of the angular deviation due to wobble effects, which, as a first approximation, shall be taken as—

- (A) for sheathing containing tendons other than bars and having an internal diameter—
 - (1) ≤ 50 mm: 0.024 to 0.016 rad/m;
 - (2) > 50 mm but ≤ 90 mm: 0.016 to 0.012 rad/m;
 - (3) > 90 mm but ≤ 140 mm: 0.012 to 0.008 rad/m.
- (B) for flat metal ducts containing tendons other than bars: 0.024 rad/m to 0.016 rad/m;
- (C) for sheathing containing bars and having an internal diameter of 50 mm or less: 0.016 rad/m to 0.008 rad/m; and
- (D) for bars of any diameter in a greased-and-wrapped coating: 0.008 rad/m.

L_{pa} = length of the tendon from the jacking end to a point at a distance (a) from that end

The magnitude of the friction due to duct curvature and wobble used in the design shall be verified during the stressing operation.

3.4.2.5 Loss of prestress during anchoring

In a post-tensioned member, allowance shall be made for loss of prestress when the prestressing force is transferred from the tensioning equipment to the anchorage. This allowance shall be checked on the site and any correspondingly required adjustment shall be made.

3.4.2.6 *Loss of prestress due to other considerations*

Where applicable, loss of prestress due to the following shall be taken into account in design:

- (a) Deformation of the forms for precast members.
- (b) Differences in temperature between stressed tendons and the actual stressed structures during heat treatment of the concrete.
- (c) Changes in temperature between the time of stressing the tendons and the time of casting concrete.
- (d) Deformations in the construction joints of precast structures assembled in sections.
- (e) Permanently elevated temperatures in excess of 40°C.

3.4.3 *Time-dependent losses of prestress*

3.4.3.1 *General*

The total time-dependent loss of prestress shall be estimated by adding the calculated losses of prestress due to shrinkage of the concrete, creep of the concrete, tendon relaxation, and other considerations (see Clause 3.4.3.5) as are applicable.

3.4.3.2 *Loss of prestress due to shrinkage of the concrete*

The loss of stress in the tendon due to shrinkage of the concrete shall be taken as $E_p \varepsilon_{cs}$, modified to allow for the effects of reinforcement, provided the shrinkage restraint effects of the reinforcement are included in the serviceability design of the member, where ε_{cs} is determined in accordance with Clause 3.1.7.2.

Where reinforcement is distributed throughout the member so that its effect on shrinkage is mainly axial, the loss of prestress in the tendons shall be taken as:

$$\frac{E_p \varepsilon_{cs}}{1 + 15 A_s / A_g} \quad \dots 3.4.3.2$$

3.4.3.3 *Loss of prestress due to creep of the concrete*

The loss of prestress due to creep of the concrete shall be calculated from an analysis of the creep strains in the concrete. In the absence of more detailed calculations and provided the sustained stress in the concrete at the level of the tendons at no time exceeds $0.5 f'_c$, the loss of stress in the tendon due to creep of the concrete shall be taken as $E_p \varepsilon_{cc}$, in which ε_{cc} is given by—

$$\varepsilon_{cc} = 0.8 \varphi_{cc} (\sigma_{ci} / E_c) \quad \dots 3.4.3.3$$

where

φ_{cc} = design creep coefficient calculated in accordance with Clause 3.1.8.3

σ_{ci} = sustained stress in the concrete at the level of the centroid of the tendons, calculated using the initial prestressing force prior to any time-dependent losses and the sustained portions of all the service loads

3.4.3.4 *Loss of prestress due to tendon relaxation*

The loss of stress in a tendon due to relaxation of the tendon in the member shall be determined by modifying the percentage loss of stress due to the design relaxation of the tendon (R) to take into account the effects of shrinkage and creep.

In the absence of more detailed calculations, the percentage loss of stress in the tendon in the member shall be taken as—

$$R \left(1 - \frac{\text{the loss of stress due to creep and shrinkage}}{\sigma_{pi}} \right) \quad \dots 3.4.3.4$$

where

σ_{pi} = stress in the tendon immediately after transfer

3.4.3.5 *Loss of prestress due to other considerations*

Account shall be taken, if applicable, of—

- (a) losses due to deformations in the joints of precast structures assembled in sections; and
- (b) losses due to the effects of any increase in creep caused by frequently repeated loads.

3.5 MATERIAL PROPERTIES FOR NON-LINEAR STRUCTURAL ANALYSIS

Where the structure is to be analysed in design in accordance with Clauses 6.5 and 6.6, mean values of all relevant material properties shall be used in the form of the stress-strain curve for the material.

SECTION 4 DESIGN FOR DURABILITY

4.1 GENERAL

The requirements of this section apply to plain, reinforced and prestressed concrete structures and members with a design life of 50 years $\pm 20\%$.

NOTES:

- 1 More stringent requirements would be appropriate for structures with a design life in excess of 50 years (e.g. monumental structures), while some relaxation of the requirements may be acceptable for structures with a design life of less than 50 years (e.g. temporary structures).
- 2 Durability is a complex topic and conformance with these requirements may not be sufficient to ensure a durable structure.

4.2 METHOD OF DESIGN FOR DURABILITY

Durability shall be allowed for in design by determining the exposure classification in accordance with Clause 4.3 and, for that exposure classification, conforming with the requirements for concrete quality and curing, in accordance with Clauses 4.4 and 4.5. In addition—

- (a) members subject to abrasion from traffic (e.g. pavements and floors) shall satisfy the requirements of Clause 4.6;
- (b) members subject to cycles of freezing and thawing shall satisfy the requirements of Clause 4.7;
- (c) members subject to aggressive soils shall satisfy Clause 4.8;
- (d) members susceptible to damage due to alkali aggregate reaction (AAR) shall be assessed and appropriate management measures shall be taken; and

NOTE: Guidance on appropriate management measures may be found in HB 79.

- (e) members containing reinforcement and/or tendons, the chemical content restrictions of the concrete shall be in accordance with Clause 4.9 and the cover to reinforcement and tendons shall be in accordance with Clause 4.10.

4.3 EXPOSURE CLASSIFICATION

4.3.1 General

The following are applicable:

- (a) The exposure classification for a surface of a member shall be determined from Table 4.3 and Figure 4.3.
- (b) For determining concrete quality requirements in accordance with Clauses 4.4 to 4.8, the exposure classification for the member shall be taken as the most severe exposure of any of its surfaces.
- (c) For determining cover requirements for corrosion protection in accordance with Clause 4.10.3, the exposure classification shall be taken as the classification for the surface from which the cover is measured.
- (d) Members that do not contain reinforcement shall have an exposure classification of A1, unless the environment is aggressive to the concrete [see also Clause 4.2, Items (a), (b), (c) and (d)].

4.3.2 Concession for exterior exposure of a single surface

Where the exterior exposure is essentially only one surface of a member, concrete of the next lower grade than would otherwise be required by Clause 4.4 may be used, provided the cover from that surface is increased by—

- (a) 20 mm from the value required by Clause 4.10.3.2; or
- (b) 15 mm from the value required by Clause 4.10.3.3.

TABLE 4.3
EXPOSURE CLASSIFICATIONS

Surface and exposure environment	Exposure classification reinforced or prestressed concrete members (see Note 1)
1 Surface of members in contact with the ground:	
(a) Members protected by a damp-proof membrane.	A1
(b) Residential footings in non-aggressive soils.	A1
(c) Other members in non-aggressive soils.	A2
(d) Members in aggressive soils:	
(i) Sulfate bearing (magnesium content <1g/L).	See Table 4.8.1
(ii) Sulfate bearing (magnesium content ≥1g/L) (see Note 2).	U
(iii) Other.	U
(e) Salt rich soils and soils in areas affected by salinity.	See Table 4.8.2
2 Surfaces of members in interior environments:	
(a) Fully enclosed within a building except for a brief period of weather exposure during construction:	
(i) Residential.	A1
(ii) Non-residential.	A2
(b) In industrial buildings, the member being subject to repeated wetting and drying.	B1
3 Surfaces of members in above-ground exterior environments in areas that are:	
(a) Inland (>50 km from coastline) environment being:	
(i) Non-industrial and arid climatic zone (see Note 3).	A1
(ii) Non-industrial and temperate climatic zone.	A2
(iii) Non-industrial and tropical climatic zone.	B1
(iv) Industrial (see Note 4) and any climatic zone.	B1
(b) Near-coastal (1 km to 50 km from coastline), any climatic zone.	B1
(c) Coastal (see Note 5) and any climatic zone.	B2
4 Surfaces of members in water:	
(a) In freshwater (continuously).	B1
(b) In soft or running water (see Note 6).	U

(continued)

TABLE 4.3 (continued)

Surface and exposure environment	Exposure classification reinforced or prestressed concrete members (see Note 1)
5 Surfaces of maritime structures in sea water:	
(a) Permanently submerged.	B2
(b) In spray zone (see Note 7).	C1
(c) In tidal/splash zone (see Note 8).	C2
6 Surfaces of members in other environments, that is, any exposure environment not specified in Items 1 to 5 above (see Note 9).	U

NOTES:

- 1 In this context, reinforced concrete includes any concrete containing metals that rely on the concrete for protection against environmental degradation. Plain concrete members containing metallic embedments should be treated as reinforced members when considering durability.
- 2 Severity of sulfate attack depends on the type of sulfate. For example, magnesium sulfate is more aggressive than sodium sulfate. The use of sulfate-resisting cement and concrete would be adequate for sodium sulfate conditions. For the magnesium sulfate conditions, specific consideration should be given to the cement and concrete that are likely to resist this type of sulfate. For magnesium sulfate soil conditions in which the concentration of magnesium is more than 1000 ppm, special consideration shall be given to the cement and concrete and other protective measures that will be required to resist this type of sulfate attack.
- 3 The climatic zones referred to are those given in Figure 4.3, which is based on the Bureau of Meteorology map, *Major seasonal rainfall zones of Australia*, Commonwealth of Australia, 2005.
- 4 Industrial refers to areas that are within 3 km of industries that discharge atmospheric pollutants.
- 5 For the purpose of this table, the coastal zone includes locations within 1 km of the shoreline of large expanses of saltwater. Where there are strong prevailing winds or vigorous surf, the distance should be increased beyond 1 km and higher levels of protection should be considered.
- 6 Exposure classification guidance based on Langelier saturation index and pH can be obtained from AS 3735.
- 7 The spray zone is the zone from 1 m above wave crest level.
- 8 The tidal/splash zone is immediately below the spray zone and includes the zone 1 m below lowest astronomical tide (LAT) and up to 1 m above highest astronomical tide (HAT) on all exposed surfaces, and all exposed soffits of horizontal structures over the sea.
- 9 Further guidance on measures appropriate in exposure classification U may be obtained from AS 3735.
- 10 In this table, classifications A1, A2, B1, B2, C1 and C2 represent increasing degrees of severity of exposure, while classification U represents an exposure environment not specified in this table but for which a degree of severity of exposure should be assessed. Protective surface coatings may be taken into account in the assessment of the exposure classification.

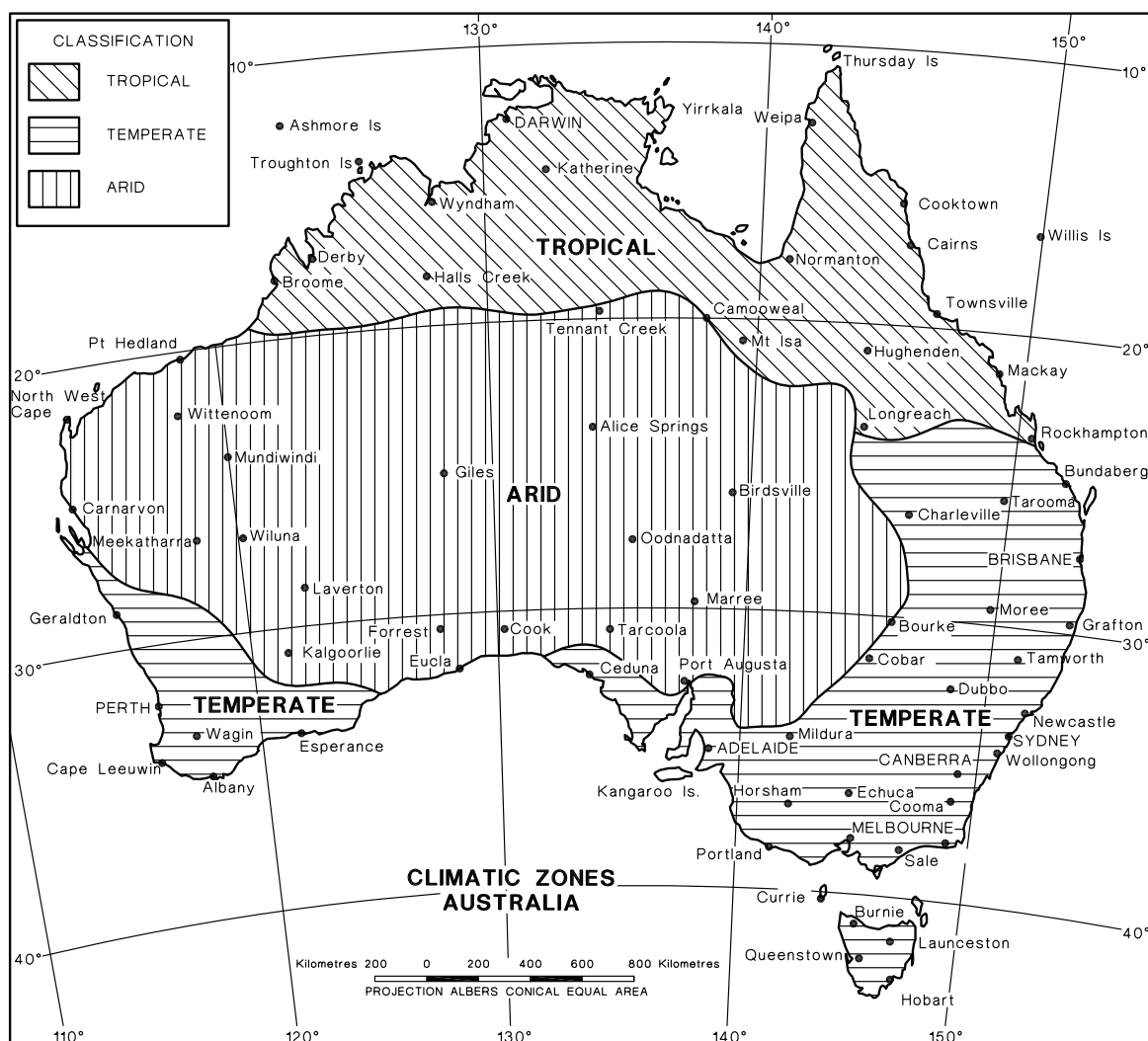


FIGURE 4.3 CLIMATIC ZONES REFERRED TO IN TABLE 4.3

4.4 REQUIREMENTS FOR CONCRETE FOR EXPOSURE CLASSIFICATIONS A1, A2, B1, B2, C1 AND C2

Members subject to exposure classifications A1, A2, B1, B2, C1 and C2 shall have minimum f'_c as specified in Table 4.4, Column 2, and be cured as specified in Table 4.4, Column 3, or have a minimum average compressive strength of the concrete at the time of stripping of forms or removal from moulds as specified in Table 4.4, Column 4.

All concrete subject to exposure classifications B2, C1 or C2 shall be specified as special class as per AS 1379 and include in the specified parameters the exposure classification and any limitations on concrete quality.

TABLE 4.4
MINIMUM STRENGTH AND CURING REQUIREMENTS FOR CONCRETE

Column 1	Column 2	Column 3	Column 4
Exposure classification	Minimum f'_c (MPa)	Minimum initial curing requirement (see Clause 17.1.5.1)	Minimum average compressive strength at the time of stripping of forms or removal from moulds (MPa)
A1	20	Cure continuously for at least 3 days	15
A2	25		
B1	32	Cure continuously for at least 7 days	20
B2	40		25
C1	50		32
C2	50		

4.5 REQUIREMENTS FOR CONCRETE FOR EXPOSURE CLASSIFICATION U

Members subject to exposure classification U shall have the concrete quality, cover to reinforcement/tendons, and other parameters specified where required, to ensure durability under the particular exposure environment.

4.6 ABRASION

In addition to the other durability requirements of this section, concrete for members subject to abrasion from traffic shall have a characteristic compressive strength not less than the applicable value given in Table 4.6.

TABLE 4.6
STRENGTH REQUIREMENTS FOR ABRASION

Member and/or traffic	Minimum characteristic compressive strength (f'_c) MPa
Footpaths and residential driveways	20
Commercial and industrial floors not subject to vehicular traffic	25
Pavements or floors subject to:	
(a) Pneumatic-tyred traffic	32
(b) Non-pneumatic-tyred traffic	40
(c) Steel-wheeled traffic	To be assessed but not less than 40

NOTE: f'_c refers to the characteristic compressive strength of the wearing surface.

4.7 FREEZING AND THAWING

In addition to the other durability requirements of this section, where the surface exposure includes exposure to cycles of freezing and thawing, concrete in the member shall—

- (a) have an f'_c not less than—
 - (i) 32 MPa for occasional exposure (<25 cycles p.a.); or
 - (ii) 40 MPa for frequent exposure (≥25 cycles p.a.); and

- (b) contain a percentage of entrained air not outside the following ranges—
- (i) for 10 mm to 20 mm nominal size aggregate..... 8% to 4%; or
 - (ii) for 40 mm nominal size aggregate6% to 3%,
- where the percentage of entrained air is determined in accordance with AS 1012.4.

4.8 AGGRESSIVE SOILS

4.8.1 Sulfate and acid sulfate soils

In addition to the other durability requirements of this section, where the surface exposure includes exposure to acid sulfate soils or to sulfate soils with a magnesium content of less than 1000 ppm, the exposure classification shall be as shown in Table 4.8.1.

TABLE 4.8.1
EXPOSURE CLASSIFICATION FOR CONCRETE IN SULFATE SOILS

Exposure conditions			Exposure classification	
Sulfates (expressed as SO ₄)*		pH	Soil conditions A†	Soil conditions B‡
In soil ppm	In groundwater ppm			
<5000	<1000	>5.5	A2	A1
5000–10 000	1000–3000	4.5–5.5	B1	A2
10 000–20 000	3000–10 000	4–4.5	B2	B1
>20 000	>10 000	<4	C2	B2

* Approximately 100 ppm SO₄ = 80 ppm SO₃.

† Soil conditions A—high permeability soils (e.g. sands and gravels) which are in groundwater.

‡ Soil conditions B—low permeability soils (e.g. silts and clays) or all soils above groundwater.

NOTES:

- 1 This is a simplistic and sometimes conservative approach to the definition of aggressivity. It is common to find more than one chemical in the service environment and the effect of these chemicals may be modified in the presence of others. For example, sulfate ions become aggressive at levels of 600 ppm to 1000 ppm when combined with magnesium or ammonium ions. In the presence of chloride ions, however, attack by sulfate ions generally exhibits little disruptive expansion with the exception of conditions of wetting and extreme drying where crystallization can cause surface fretting of concrete.
- 2 Chemical concentrations relate only to the proportion of chemical present that is water-soluble.
- 3 Where exposure classifications 'B1', 'B2', 'C1' or 'C2' are indicated, it is recommended that the cement be Type SR.
- 4 Where exposure classifications 'B2', 'C1' or 'C2' are in acid sulfate soil conditions, it is recommended that a protective coating is used on the concrete surface.
- 5 Where a protective coating is used it may be possible to reduce the minimum required reinforcement cover to 50 mm.
- 6 Acidic ground conditions can be caused by dissolved 'aggressive' carbon dioxide, pure and very soft waters, organic and mineral acids and bacterial activity. Care is required in assessment of pH underground structure and lifetime conditions since pH can change over the lifetime of the member. Therefore, the pH should not be assessed only on the basis of a present-day test result, rather the ground chemistry should be considered over the design life of the ground structure. Testing for pH should be carried out either in situ or immediately after sampling as there is otherwise a risk of oxidation with time, leading to apparent acidity, which does not correctly represent in situ conditions.
- 7 pH alone may be a misleading measure of aggressivity without a full analysis of causes (e.g. still versus running water).
- 8 Contamination by the tipping of mineral and domestic wastes or by spillage from mining, processing or manufacturing industries presents special durability risks due to the presence of certain aggressive acids, salts and solvents, which can either chemically attack concrete or lead to a corrosion risk. Certain ground conditions cannot be properly addressed by reference only to Table 4.8.1. These conditions include, for example, areas where acid-sulfate soils exist, contamination by industrial and domestic waste, or spillage from mining, processing or manufacturing industries. In the absence of site-specific chemical information, the exposure condition should be assessed as 'exposure classification B2' for domestic refuse and 'exposure classification C2' for industrial/mining waste tips. Chemical analysis of the latter may, however, allow a lower risk classification.

The minimum reinforcement cover shall be the greater of the values given in Tables 4.8.2, 4.10.3.2 and 4.10.3.3, as varied by Clause 4.10.3.5 for the appropriate exposure classification, placement conditions and characteristic strength of concrete.

4.8.2 Saline soils

Where the surface exposure includes exposure to saline soils, the minimum requirement shall be as shown in Table 4.8.2.

TABLE 4.8.2
STRENGTH AND COVER REQUIREMENTS FOR SALINE SOILS

Soil electrical conductivity (EC_e)	Exposure classification	Minimum f'_c	Minimum cover (mm)
4–8	A2	25	45
8–16	B1	32	50
>16	B2	40	55

NOTES:

- 1 EC_e is saturated electrical conductivity in deciSiemens per metre.
- 2 Guidance on concrete in saline environments can be found in CCAA T56.

4.9 RESTRICTIONS ON CHEMICAL CONTENT IN CONCRETE

Certain chemical constituents (e.g. chlorides) can have deleterious effects on the durability of concrete. For this reason, chemical admixtures added to concrete to be used in structures or members designed in accordance with this Standard shall conform with AS 1478.1 and chemical content in concrete shall conform with AS 1379.

4.10 REQUIREMENTS FOR COVER TO REINFORCING STEEL AND TENDONS

4.10.1 General

The cover to reinforcing steel and tendons shall be the greatest of the values determined in accordance with Clauses 4.8, 4.10.2 and 4.10.3, unless exceeded by the requirements for fire resistance given in Section 5.

4.10.2 Cover for concrete placement

Designers shall specify appropriate covers to ensure the concrete can be satisfactorily placed and compacted around the reinforcement, tendons or ducts, or any combination of these, in accordance with the requirements of Clause 17.1.3.

A2 | In the determination of cover, consideration shall be given to—

- (a) the size and shape of the member;
- (b) the size, type and configuration of the reinforcement and, if present, the tendons or ducts; and
- (c) the aggregate size, the workability of the concrete and the direction of concrete placement.

Where the presence of ducts is not a consideration, covers to reinforcement or tendons greater than their nominal size or the maximum nominal aggregate size, whichever is larger, shall be deemed to satisfy the requirements of the first two paragraphs of the clause.

4.10.3 Cover for corrosion protection

4.10.3.1 General

For corrosion protection, the cover shall be not less than the value given in accordance with Clauses 4.10.3.2 to 4.10.3.7.

A2

4.10.3.2 Required cover where standard formwork and compaction are used

Where concrete is cast in formwork conforming with AS 3610 and compacted in accordance with Clause 17.1.3 of this Standard, the cover shall be not less than the value given in Tables 4.8.1, 4.8.2 and 4.10.3.2, as per the exposure classification and f'_c .

TABLE 4.10.3.2
REQUIRED COVER WHERE STANDARD FORMWORK
AND COMPACTION ARE USED

Exposure classification	Required cover, mm				
	Characteristic strength (f'_c)				
	20 MPa	25 MPa	32 MPa	40 MPa	≥ 50 MPa
A1	20	20	20	20	20
A2	(50)	30	25	20	20
B1	—	(60)	40	30	25
B2	—	—	(65)	45	35
C1	—	—	—	(70)	50
C2	—	—	—	—	65

NOTE: Bracketed figures are the covers when the concession given in Clause 4.3.2, relating to the strength grade permitted for a particular exposure classification, is applied.

4.10.3.3 Required cover where repetitive procedures or intense compaction are used in rigid formwork

Where concrete members are cast in rigid formwork under repetitive procedures, with demonstrated process control systems including supervision, the cover shall be not less than the value given in Tables 4.8.1, 4.8.2 and 4.10.3.3, as per the exposure classification and f'_c . Cover to uniformed surfaces of members shall be in accordance with Table 4.10.3.2.

TABLE 4.10.3.3
REQUIRED COVER WHERE REPETITIVE PROCEDURES
AND INTENSE COMPACTION OR SELF-COMPACTING
CONCRETE ARE USED IN RIGID FORMWORK

Exposure classification	Required cover, mm				
	Characteristic strength (f'_c)				
	20 MPa	25 MPa	32 MPa	40 MPa	≥ 50 MPa
A1	20	20	20	20	20
A2	(45)	30	20	20	20
B1	—	(45)	30	25	20
B2	—	—	(50)	35	25
C1	—	—	—	(60)	45
C2	—	—	—	—	60

NOTE: Bracketed figures are the covers when the concession given in Clause 4.3.2, relating to the strength grade permitted for a particular exposure classification, is applied.

4.10.3.4 *Required cover where self-compacting concrete is used*

Where concrete members are cast with self-compacting concrete, the cover shall be not less than the value given in Tables 4.8.1, 4.8.2 and 4.10.3.3. Screeded surfaces of members shall be in accordance with Table 4.10.3.2.

4.10.3.5 *Cast against ground*

Where concrete is cast on or against the ground and compacted in accordance with Clause 17.1.3, the cover to a surface in contact with the ground shall be as given in Table 4.10.3.2 but increased by—

- (a) 10 mm if the concrete surface is protected by a damp-proof membrane; or
- (b) 20 mm otherwise.

4.10.3.6 *Structural members manufactured by spinning or rolling*

Where structural members are manufactured by spinning and rolling concrete, the cover for corrosion protection shall be as specified in the appropriate Standard, where an equivalent exposure classification (as identified in Clause 4.3.1 of this Standard) for the required design life can be demonstrated.

4.10.3.7 *Embedded items cover*

A1

Embedded items, as defined in Clause 19.2, shall be protected from corrosion or deterioration. The cover to embedded items that are not corrosion resistant shall be as given in Table 4.10.3.2 and Table 4.10.3.3, as applicable.

Metals such as aluminium shall not be embedded in structural concrete unless effectively coated, covered, or treated to prevent chemical action between the metal and the concrete and electrolytic action between the metal and steel.

SECTION 5 DESIGN FOR FIRE RESISTANCE

5.1 SCOPE

This section specifies the requirements for reinforced and prestressed concrete members used in buildings to meet the fire resistance levels (FRLs) required by the Building Code of Australia (BCA).

5.2 DEFINITIONS

For the purpose of this section, the definitions below apply.

5.2.1 Average axis distance

When reinforcement is arranged in several layers as shown in Figure 5.2.1, and where it consists of either reinforcement or prestressing tendons with the same characteristic strength f_{sy} and f_{py} respectively, the average axis distance (a_m) may be determined by—

$$a_m = \frac{A_{s1}a_1 + A_{s2}a_2 + \dots + A_{sn}a_n}{A_{s1} + A_{s2} + \dots + A_{sn}} = \frac{\sum A_{si}a_i}{\sum A_{si}} \quad \dots 5.2.1$$

where

A_{si} = cross-sectional area of steel bar (tendon, wire) ‘i’

a_i = axis distance of steel bar (tendon, wire) ‘i’ from the nearest exposed surface

When reinforcement consists of steels with different characteristic strength, A_{si} should be replaced by $A_{si}f_{syi}$ (or $A_{si}f_{pyi}$) in the above equation.

Where reinforcement and prestressing tendons are used simultaneously (e.g. in a partially prestressed member), the axis distances of reinforcement and prestressing tendons shall be determined separately.

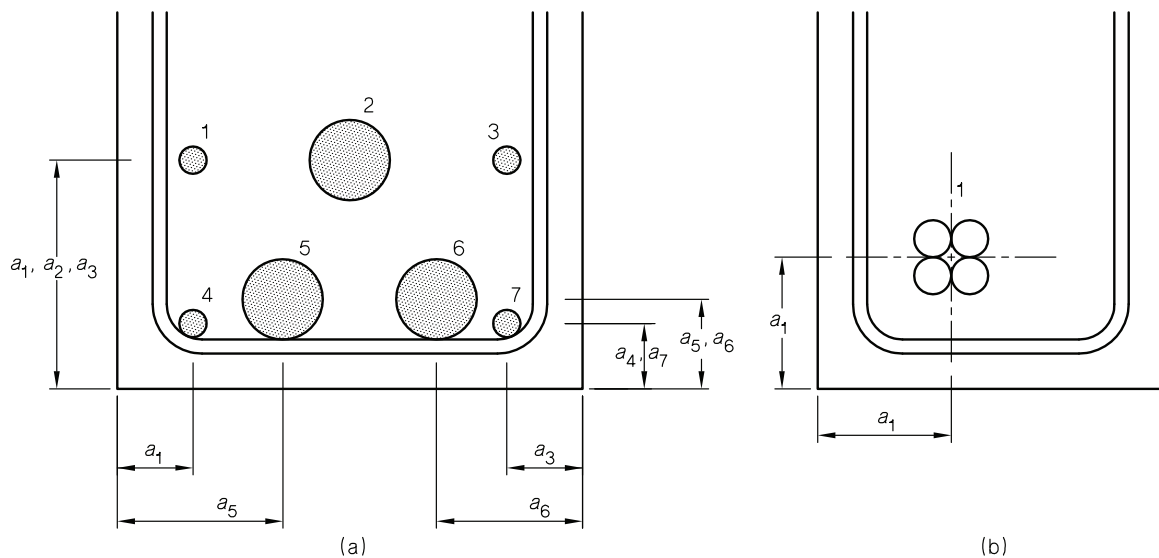
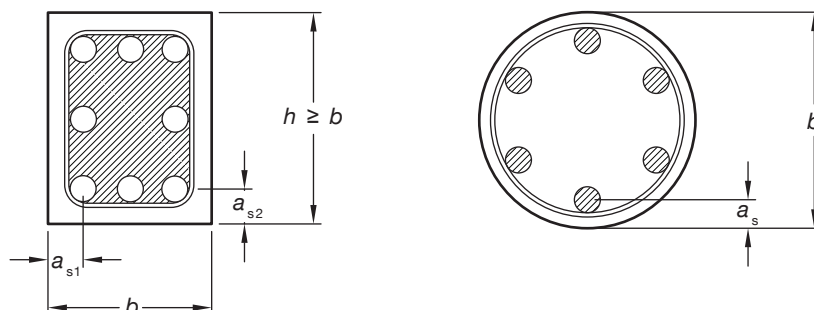


FIGURE 5.2.1 DIMENSIONS USED TO CALCULATE AVERAGE AXIS DISTANCE

5.2.2 Axis distance

Distance from the centre-line axis of a longitudinal bar or tendon to the nearest surface exposed to fire (see Figure 5.2.2).



NOTE: Axis distance (a_s) is a nominal value and no allowance for tolerance need be added.

FIGURE 5.2.2 SECTIONS THROUGH STRUCTURAL MEMBERS SHOWING AXIS DISTANCE (a_s)

5.2.3 Fire resistance

Ability of a structure or part of it to fulfil its required functions (loadbearing and/or separating function) for a specified fire exposure, for a specified time.

5.2.4 Fire resistance level (FRL)

Fire resistance periods for structural adequacy, integrity and insulation, expressed in that order.

NOTE: Fire resistance levels for structures, parts and elements of construction are specified by the relevant authority, for example, in the Building Code of Australia (BCA).

5.2.5 Fire resistance period (FRP)

Time, in minutes, for a member to reach the appropriate failure criteria (i.e. structural adequacy, integrity and/or insulation) if tested for fire in accordance with the appropriate Standard.

NOTE: Where the Building Authority references the Building Code of Australia, the appropriate Standard is AS 1530.4.

5.2.6 Fire-separating function

Ability of a boundary element of a fire compartment (e.g. wall, floor or roof) to prevent fire spread by passage of flames or hot gases (integrity) or ignition beyond the exposed surface (thermal insulation) during a fire.

NOTE: When tested in accordance with AS 1530.4, prototypes of such members are exposed to fire from only one direction at a time and are assumed to be similarly exposed for the purpose of interpreting Section 5.

5.2.7 Insulation (fire)

The ability of a fire-separating member, such as a wall or floor, to limit the surface temperature on one side of the member when exposed to fire on the other side.

5.2.8 Integrity (fire)

Ability of a fire-separating member to resist the passage of flames or hot gases through the member when exposed to fire on one side.

5.2.9 Structural adequacy (fire)

Ability of a member to maintain its structural function when exposed to fire.

5.3 DESIGN PERFORMANCE CRITERIA

5.3.1 General performance criteria

A member shall be designed to have a fire resistance period (FRP) for structural adequacy, integrity and insulation of not less than the required fire resistance level (FRL).

If applicable, the criteria for integrity shall be considered to be satisfied if the member meets the criteria for both insulation and structural adequacy for that period.

The FRP for a member shall be established by either one of the following methods:

- (a) Determined from the tabulated data and figures given in this section. Unless stated otherwise within this section, when using the tabulated data or figures no further checks are required concerning shear and torsion capacity or anchorage details.
- (b) Predicted by methods of calculation. In these cases, checks shall be made for bending, and shear, torsion and anchorage capacities as required.

NOTES:

- 1 Eurocode 2, Part 1.2 provides a method of calculation to predict the FRP of a member.
- 2 Eurocode 2 Part 1.2 Amendment 1 Annex C provides tabulated data for the fire design of a range of slender reinforced concrete columns in braced and unbraced structures. The data in these tables is based on the calculation methods recommended in Note 1.

5.3.2 General rules for the interpretation of tabulated data and figures

Linear interpolation between values given in the tables and figures in this section is permitted. Values in the tables provide minimum dimensions for fire resistance. Some values of the axis distance of the reinforcement or tendons will result in covers less than those required for durability or compaction and are provided only to allow interpolation within the table or figure.

5.3.3 Increase in axis distance for prestressing tendons

The required axis distance for prestressing tendons shall be that determined for reinforcing bars as shown in the tables and figures of this section, increased by—

- (a) 15 mm for prestressing strand or wire; or
- (b) 10 mm for prestressing bars.

5.3.4 Dimensional limitations to achieve fire-rating

Where hollow-core slabs or walls are required to achieve a FRL, the thickness of concrete between adjacent voids and the thickness of concrete between any part of a void and the nearest surface shall be not less than the greater of one fifth the required effective thickness of the slab or wall and 25 mm.

Where ribbed slabs are required to achieve a FRL, the ribs shall be spaced at not greater than 1500 mm centre-to-centre.

5.3.5 Joints

Joints between members or between adjoining parts shall be constructed so that the FRL of the whole assembly is not less than that required for the member.

5.3.6 The effect of chases

In concrete members subject to fire, chases shall be kept to a minimum. The effect of chases on the FRPs of walls shall be taken into account in accordance with the requirements

of Clause 5.7.4. The effect of chases in other members shall be taken into account using a rational method of analysis.

5.3.7 Increasing FRPs by the addition of insulating materials

The FRP for insulation and structural adequacy of a concrete member may be increased by the addition to the surface of an insulating material, to provide increased thickness to the member or greater insulation to the longitudinal reinforcement or tendons, or both, in accordance with the requirements of Clause 5.8.

For slabs, the FRPs may be increased by the addition of toppings and/or the application of insulating materials to the soffit; for flat slabs and plates, only the application of an insulating material to the soffit may be used to improve the structural adequacy.

For walls, the FRPs may be increased by the application of insulating materials to the face exposed to fire.

In either case, other methods (e.g. addition of insulation materials in hollow cores) may be used. Any increase afforded shall be determined in accordance with Clause 5.8.

5.4 FIRE RESISTANCE PERIODS (FRPs) FOR BEAMS

5.4.1 Structural adequacy for beams incorporated in roof or floor systems

The FRP for structural adequacy for a beam incorporated in a roof or floor system is given by—

- (a) Table 5.4.1(A) or Figure 5.4.1(A) for simply supported beams; or
 - (b) Table 5.4.1(B) or Figure 5.4.1(B) for continuous beams;
- provided the beam—
- (i) has the upper surface integral with or protected by a slab conforming with Clause 5.5;
 - (ii) has a web of uniform width, or one which tapers uniformly over its depth; and
 - (iii) is proportioned so that—
 - (A) the beam width (b), measured at the centroid of the lowest level of longitudinal bottom reinforcement; and
 - (B) the average axis distance to the longitudinal bottom reinforcement, are not less than the values for that period given in the appropriate table or figure.

For the purpose of this clause, a beam shall be considered continuous if, under imposed actions, it is designed as flexurally continuous at one or both ends.

TABLE 5.4.1(A)
FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY
FOR SIMPLY SUPPORTED BEAMS

FRP for structural adequacy (min)	Minimum dimensions (mm)							
	Possible combinations of a_m and b							
	Combination 1		Combination 2		Combination 3		Combination 4	
	a_m	b	a_m	b	a_m	b	a_m	b
30	25	80	20	120	15	160	15	200
60	40	120	35	160	30	200	25	300
90	55	150	45	200	40	300	35	400
120	65	200	60	240	55	300	50	500
180	80	240	70	300	65	400	60	600
240	90	280	80	350	75	500	70	700

LEGEND:

a_m = average axis distance

b = width of the beam at the centroid of the bottom reinforcement

NOTES:

- 1 In beams with only one layer of bottom reinforcement, the axis distance to the side of the beam for the corner bars including tendons or wires, shall be increased by 10 mm, except, where the value of b is greater than that given in Combination 4, no increase is required.
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

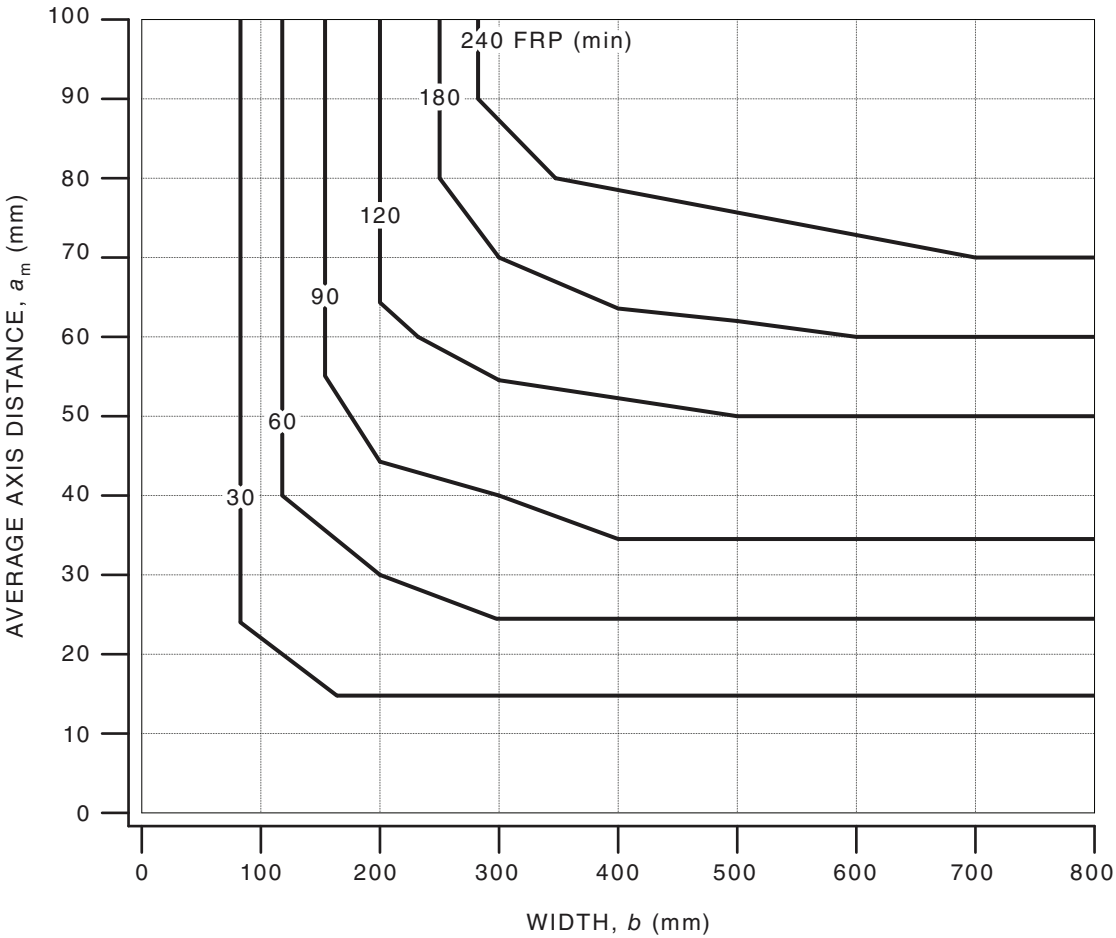


FIGURE 5.4.1(A) FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY
FOR SIMPLY SUPPORTED BEAMS

TABLE 5.4.1(B)
FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY
FOR CONTINUOUS BEAMS

FRP for structural adequacy (min)	Minimum dimensions (mm)							
	Possible combinations of a_s and b							
	Combination 1		Combination 2		Combination 3		Combination 4	
	a_m	b	a_m	b	a_m	b	a_m	b
30	15	80	12	160	—	—	—	—
60	25	120	12	200	—	—	—	—
90	35	150	25	250	—	—	—	—
120	45	200	35	300	35	450	30	500
180	60	240	50	400	50	550	40	600
240	75	280	60	500	60	650	50	700

LEGEND:

a_m = average axis distance

b = width of the beam at the centroid of the bottom reinforcement

NOTES:

- 1 In beams with only one layer of bottom reinforcement, the axis distance to the side of the beam for the corner bars including tendons or wires, shall be increased by 10 mm, except, where the value of b is greater than that given in Combination 4 no increase is required.
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

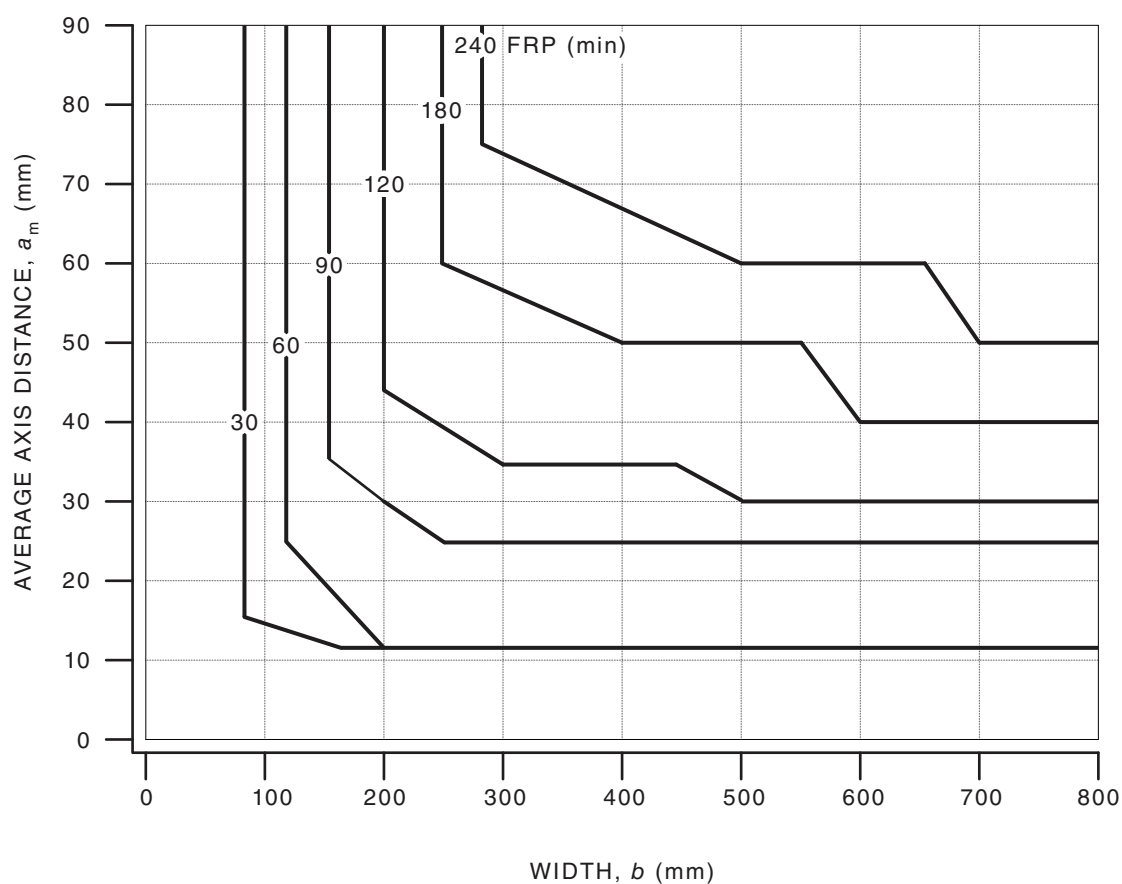


FIGURE 5.4.1(B) FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY FOR CONTINUOUS BEAMS

5.4.2 Structural adequacy for beams exposed to fire on all sides

The FRP for structural adequacy for a beam of approximately rectangular cross-section, which can be exposed to fire on all four sides, is given by—

- (a) Table 5.4.1(A) or Figure 5.4.1(A) for simply supported beams; or
- (b) Table 5.4.1(B) or Figure 5.4.1(B) for continuous beams, provided in each case the beam is proportioned so that—
 - (i) the total depth of the beam is not less than the least value of b for that period;
 - (ii) the cross-sectional area of the beam is not less than twice the area of a square with a side equal to b determined as for Item (a); and
 - (iii) the average axis distance is not less than the value for that period determined using the minimum dimension of the beam for b in the relevant table and applies to all longitudinal reinforcement or tendons.

5.5 FIRE RESISTANCE PERIODS (FRPs) FOR SLABS

5.5.1 Insulation for slabs

The FRP for insulation for a slab is given in Table 5.5.1 provided the effective thickness of the slab is not less than the corresponding value given in the table.

The effective thickness of the slab to be used in Table 5.5.1 shall be taken as—

- (a) for solid slabs, the actual thickness; or
- (b) for hollow-core slabs, the net cross-sectional area divided by the width of the cross-section; or
- (c) for ribbed slabs, the thickness of the solid slab between the webs of adjacent ribs.

TABLE 5.5.1
FIRE RESISTANCE PERIODS (FRPs)
FOR INSULATION FOR SLABS

FRP for insulation min	Effective thickness mm
30	60
60	80
90	100
120	120
180	150
240	175

5.5.2 Structural adequacy for slabs

The FRP for structural adequacy for a slab shall be deemed to be satisfied if the following conditions are met:

- (a) For solid or hollow-core slabs supported on beams or walls [see Table 5.5.2(B)], provided the slab is proportioned such that, for the appropriate support conditions, the average axis distance to the bottom reinforcement and tendons is not less than the value for that period given in the Table.
- (b) For flat slabs, including flat plates [see Table 5.5.2(A)], provided—
 - (i) the average axis distance to the bottom layer of reinforcement and tendons is not less than the value in the table; and

- (ii) The minimum slab thickness defined in Table 5.5.2(A) is required over a rectangular area extending 0.16 of the larger span length either side of a column in each direction from the column face. The depths in this table cannot be reduced by calculation methods in accordance with Clause 5.3.1(b). Outside this area, the minimum slab depths for insulation in Table 5.5.1 for insulation apply.
- (c) For one-way ribbed slabs, see Table 5.5.2(B) for the appropriate support conditions, if the slab is proportioned so that—
 - (i) the width of the ribs and the axis distance to the lowest layer of the longitudinal bottom reinforcement in the slabs conform with the requirements for beams given in Clause 5.4.1; and
 - (ii) the axis distance to the bottom reinforcement in the slab between the ribs is not less than that given in Table 5.5.2(B).
- (d) For two-way ribbed slabs, see Table 5.5.2(C) or Table 5.5.2(D) as appropriate for the support conditions. The slabs shall be proportioned so the width and the average axis distance to the longitudinal bottom reinforcement in the ribs, and the axis distance to the bottom reinforcement in the slab between the ribs, and the axis distance of the corner bar to the side face of the rib, is not less than that value plus 10 mm.

For the purpose of this clause, a slab shall be considered continuous if, under imposed actions, it is designed as flexurally continuous at one or both ends.

TABLE 5.5.2(A)
FIRE RESISTANCE PERIODS (FRPs)
FOR STRUCTURAL ADEQUACY
FOR FLAT SLABS INCLUDING FLAT PLATES

FRP for structural adequacy (min)	Minimum dimensions (mm)	
	Slab thickness	Axis distance (a_s)
30	150	10
60	180	15
90	200	25
120	200	35
180	200	45
240	200	50

NOTES:

- 1 a_s = axis distance to the reinforcement in the lowest layer.
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

TABLE 5.5.2(B)
FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY
FOR SOLID AND HOLLOW-CORE SLABS SUPPORTED ON BEAMS
OR WALLS AND FOR ONE-WAY RIBBED SLABS

FRP for structural adequacy (min)	Axis distance (a_s) to lowest layer of reinforcement (mm)			
	Simply supported slabs			Continuous slabs (one-way and two-way)
	One-way	Two-way		
		$l_y/l_x \leq 1.5$	$1.5 < l_y/l_x \leq 2$	
30	10	10	10	10
60	20	10	15	10
90	30	15	20	15
120	40	20	25	20
180	55	30	40	30
240	65	40	50	40

NOTES:

- 1 l_y = longer span of a two-way slab.
 l_x = shorter span of a two-way slab.
- 2 The axis distance for simply supported two-way slabs applies only if the slabs are supported at all four edges. In other cases, the slab shall be treated as a one-way slab.
- 3 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

TABLE 5.5.2(C)
FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY
FOR TWO-WAY SIMPLY SUPPORTED RIBBED SLABS

FRP for structural adequacy (min)	Minimum dimensions (mm)							
	Possible combinations of axis distance (a_s) and width of ribs (b)						Flange thickness (h_s) and axis distance (a_s) in flange	
	Combination 1		Combination 2		Combination 3			
	a_s	b	a_s	b	a_s	b	a_s	h_s
30	15	80	—	—	—	—	10	80
60	35	100	25	120	15	≥ 200	10	80
90	45	120	40	160	30	≥ 250	15	100
120	60	160	55	190	40	≥ 300	20	120
180	75	220	70	260	60	≥ 410	30	150
240	90	280	75	350	70	≥ 500	40	175

NOTES:

- 1 The axis distance is measured to the lowest layer of the longitudinal reinforcement.
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

TABLE 5.5.2(D)
FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY
FOR TWO-WAY CONTINUOUS RIBBED SLABS

FRP for structural adequacy (min)	Minimum dimensions (mm)							
	Possible combinations of axis distance (a_s) and width of ribs (b)						Flange thickness (h_s) and axis distance (a_s) in flange	
	Combination 1		Combination 2		Combination 3		a_s	h_s
	a_s	b	a_s	b	a_s	b		
30	10	80	—	—	—	—	10	80
60	25	100	15	120	10	≥200	10	80
90	35	120	25	160	15	≥250	15	100
120	45	160	40	190	30	≥300	20	120
180	60	310	50	600	—	—	30	150
240	70	450	60	700	—	—	40	175

NOTES:

- 1 The axis distance is measured to the lowest layer of the longitudinal reinforcement.
- 2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

5.6 FIRE RESISTANCE PERIODS (FRPs) FOR COLUMNS

5.6.1 Insulation and integrity for columns

FRPs for insulation and integrity are required for columns only where columns form part of a wall required to have a separating function. In this situation the column shall conform with the criteria for walls given in Clause 5.7.1.

5.6.2 Structural adequacy for columns

The FRP for structural adequacy for braced columns shall be determined using either Clause 5.6.3 or Clause 5.6.4. Where the ratio of the longer cross-section dimension of the column is equal to or greater than four times the shorter cross-section dimension, Clause 5.7.2 may be used.

NOTE: Clauses 5.6.3 and 5.6.4 cover only braced columns that conform with a series of restrictions. For unbraced or sway columns and braced columns outside these restrictions, see Clause 5.3.1 or the BCA and use a performance solution.

Where columns are to be designed as walls using Clause 5.7.2, the case of a wall exposed on two faces shall be adopted and the column shall be reinforced with two layers of longitudinal reinforcement (one layer located adjacent to each face), and the two layers shall be structurally restrained together.

5.6.3 Restricted tabular method to determine structural adequacy for columns

The FRP for structural adequacy for a column may be determined from Table 5.6.3, provided the following criteria are met:

- (a) The column is proportioned so that the value for the smaller cross-sectional dimension and the axis distance to the longitudinal reinforcement are not less than the values for that period.
- (b) The value of the load level is taken as 0.7 or calculated as follows:

$$\mu_{fi} = \frac{N_f^*}{\phi N_u} \quad \dots 5.6.3(1)$$

where

N_f^* = design axial load in the fire situation

N_u = ultimate strength in compression, or tension, at a cross-section of an eccentrically loaded compression or tension member respectively

- A1 | (c) Where $A_s \geq 0.02A_g$ and the required FRP is greater than 90 min, the bars are distributed along all the faces of the column.
- (d) The effective length of the column under fire conditions (metres) is less than 3 m.
- (e) The maximum eccentricity is limited to $0.15b$.
- A2 | (f) Area of longitudinal reinforcement $A_s \leq 0.04A_g$.

For columns that fall outside these limits, reference shall be made to alternative design approaches, as specified in Clause 5.3.1.

NOTE: The effective length of a column under fire conditions ($l_{0,fi}$) should be assumed to be equal to the effective length at normal temperature in all cases. For braced building structures where the required FRP is greater than 30 minutes, the effective length may be taken as $0.5L_u$ for all cases.

TABLE 5.6.3
FIRE RESISTANCE PERIODS (FRPs)
FOR STRUCTURAL ADEQUACY OF COLUMNS

FRP for structural adequacy	Minimum dimensions, mm							
	Combinations for column exposed on more than one side						Column exposed on one side	
	$\mu_{fi} = 0.2$		$\mu_{fi} = 0.5$		$\mu_{fi} = 0.7$		$\mu_{fi} = 0.7$	
	a_s	b	a_s	b	a_s	b	a_s	b
min								
30	25	200	25	200	32 27	200 300	25	155
60	25	200	36 31	200 300	46 40	250 350	25	155
90	31 25	200 300	45 38	300 400	53 40 ⁽¹⁾	350 450 ⁽¹⁾	25	155
120	40 35	250 350	45 ⁽¹⁾ 40 ⁽¹⁾	350 ⁽¹⁾ 450 ⁽¹⁾	57 ⁽¹⁾ 51 ⁽¹⁾	350 ⁽¹⁾ 450 ⁽¹⁾	35	175
180	45 ⁽¹⁾	350 ⁽¹⁾	63 ⁽¹⁾	350 ⁽¹⁾	70 ⁽¹⁾	450 ⁽¹⁾	55	230
240	61 ⁽¹⁾	350 ⁽¹⁾	75 ⁽¹⁾	450 ⁽¹⁾			70	295

LEGEND:

a_s = axis distance

b = smaller cross-sectional dimension of a rectangular column or the diameter of a circular column

NOTES:

- These combinations are for columns with a minimum of eight bars.
- For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.
- Dimension b in table for columns exposed on one side applies only to columns that lie flush with a wall having the same FRP as the column or to columns protruding from the wall, provided the part within the wall is able to carry the whole load. Openings in the wall shall not be closer to the column than the minimum dimension b for the column for the FRP. In all other cases, the column shall be treated as a column exposed on more than one side.

The FRP for structural adequacy for columns outside the limits defined for Table 5.6.3 and within the limits defined in the variables below may be assessed using Equation 5.6.3(2):

$$FRP = 120 ((R_{\eta,fi} + R_a + R_l + R_b + R_n)/120)^{1.8} \quad \dots 5.6.3(2)$$

where

$$R_{\eta,fi} = 83(1.0 - \mu_{fi}(1 + \omega)/(.945 + \omega))$$

$$R_a = 1.60 (a_s - 30)$$

$$R_l = 9.60 (5 - l_{o,fi})$$

$$R_b = 0.09 b'$$

$$R_n = 0 \text{ for 4 (corner bars only)}$$

$$= 12 \text{ for greater than 4 bars}$$

$$a_s = \text{the axis distance to the longitudinal steel bars (mm); } 25 \text{ mm} \leq a_s \leq 80 \text{ mm}$$

$$l_{o,fi} = \text{the effective length of the column under fire conditions; } 2 \text{ metres} \leq l_{o,fi} \leq 6 \text{ metres}$$

When $l_{o,fi} < 2$ m, it is conservative to take $l_{o,fi} = 2$ m in Equation 5.6.3(2)

$$b' = 2A_g / (y + x) \text{ for rectangular cross-sections}$$

$$= \text{the diameter of circular cross-sections within the limits}$$

$$200 \text{ mm} \leq b' \leq 450 \text{ mm and } y \leq 1.5x$$

$$\omega = 1.3 A_{s,fi} f_{sy} / A_g f'_c \text{ denotes the mechanical reinforcement ratio at normal temperature conditions}$$

5.6.4 General tabular method to determine structural adequacy for columns

The FRP for structural adequacy for a column may be determined from Table 5.6.4, provided the following criteria are met:

(a) The column is proportioned so that the value for the smaller cross-sectional dimension and the axial distance to the longitudinal reinforcement are not less than the values for that period.

(b) The eccentricity under fire conditions is $e = \frac{M_f^*}{N_f^*}$.

(c) $e/b < 0.25$ with $e_{\max.} \leq 100$ mm.

(d) The slenderness of the column under fire conditions is ≤ 30 .

(e) Where $A_s \geq 0.02A_g$ and $FRP > 90$ min, the reinforcing bars are assumed to be distributed evenly along the sides of the cross-section.

For columns that fall outside these limits, reference shall be made to alternative design approaches, as specified in Clause 5.3.1.

TABLE 5.6.4
FIRE RESISTANCE PERIODS (FRPs) FOR STRUCTURAL ADEQUACY
OF BRACED COLUMNS

A2

FRP for structural adequacy	$1.3 A_s f_{sy} / A_g f'_g$	Minimum dimensions, mm								
		Combinations of (a_s) and (b)								
		$\eta = 0.15$		$\eta = 0.3$		$\eta = 0.5$		$\eta = 0.7$		
		a_s	b	a_s	b	a_s	b	a_s	b	
min	30	0.1	25	150	25	150	30	200	30	300
							25	250	25	350
		0.5	25	150	25	150	25	150	30	200
									25	250
		1.0	25	150	25	150	25	150	30	200
									25	300
60	0.1	30	150	40	200	40	300	25	500	
		25	200	25	300	25	500			
	0.5	25	150	35	150	35	250	40	350	
				25	200	25	350	25	550	
	1.0	25	150	30	150	40	200	50	300	
				25	200	25	400	30	600	
90	0.1	40	200	40	300	50	500	40	550	
		25	250	25	400	25	550	25	600	
	0.5	35	150	45	200	45	300	50	500	
		25	200	25	300	25	550	40	600	
	1.0	25	200	40	200	40	250	50	500	
				25	300	25	550	45	600	
120	0.1	50	250	50	400	25	550	60	550	
		25	350	25	550			45	600	
	0.5	45	200	45	300	50	450	60	500	
		25	300	25	550	25	600	50	600	
	1.0	40	200	50	250	45	450	60	600	
		25	250	25	400	30	600			
180	0.1	50	400	60	500	60	550	(Note 1)		
		25	500	25	550	30	600			
	0.5	45	300	50	450	60	500	75	600	
		25	450	25	600	50	600			
	1.0	35	300	50	450	60	500	(Note 1)		
		25	400	25	550	45	600			

(continued)

TABLE 5.6.4 (continued)

FRP for structural adequacy	$1.3 A_s f_{sy} / A_g f'_g$	Minimum dimensions, mm							
		Combinations of (a_s) and (b)							
		$\eta = 0.15$		$\eta = 0.3$		$\eta = 0.5$		$\eta = 0.7$	
		a_s	b	a_s	b	a_s	b	a_s	b
min	0.1	60	500	40	550	75	600	(Note 1)	
		25	550	25	600				
	0.5	45	450	55	550	70	600	(Note 1)	
		25	500	25	600				
	1.0	45	400	40	500	60	600	(Note 1)	
		25	500	30	600				

LEGEND:

 a_s = axis distance b = smaller cross-sectional dimension of a rectangular column or the diameter of a circular column

$$\eta = \frac{N_f^*}{0.7 \left[\left(\frac{A_c f'_c}{1.5} \right) + \left(\frac{A_s f_{sy}}{1.15} \right) \right]}$$

NOTES:

- 1 Requires a width greater than 600 mm and assessment for buckling.
- 2 The slenderness limit of ≤ 30 is applicable to the majority of columns in normal buildings.
- 3 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.
- 4 Dimension b in table for columns exposed on one side applies only to columns that lie flush with a wall having the same FRP as the column or to columns protruding from the wall, provided the part within the wall is able to carry the whole load. Openings in the wall shall not be closer to the column than the minimum dimension b for the column for the FRP. In all other cases, the column shall be treated as a column exposed on more than one side.

5.7 FIRE RESISTANCE PERIODS (FRPs) FOR WALLS

5.7.1 Insulation for walls

The FRP for insulation for a wall given in Table 5.7.1 may be used, provided the effective thickness of the wall is not less than the corresponding value given in the table.

The effective thickness of the wall to be used in Table 5.7.1 shall be taken as follows:

- (a) For solid walls, the actual thickness.
- (b) For hollow-core walls, the net cross-sectional area divided by the length of the cross-section.

TABLE 5.7.1
FIRE RESISTANCE PERIODS (FRPs)
FOR WALLS FOR INSULATION

FRP for insulation min	Effective thickness mm
30	60
60	80
90	100
120	120
180	150
240	175

5.7.2 Structural adequacy for walls

The FRP for structural adequacy for a wall shall be in accordance with Table 5.7.2, provided the thickness of the wall and axis distance are not less than the corresponding values given in the table.

For walls where the lateral support at the top of the wall is provided on one side only by a member not required by the relevant authority to have an FRL, the structural adequacy shall be deemed to have been achieved by satisfying the requirements of Clause 5.7.1.

TABLE 5.7.2
FIRE RESISTANCE PERIODS (FRPs)
FOR STRUCTURAL ADEQUACY FOR WALLS

FRP for structural adequacy	Minimum dimensions, mm Combinations of a_s and t_w							
	$\frac{N_f^*}{\phi N_u} = 0.35$				$\frac{N_f^*}{\phi N_u} = 0.7$			
	Wall exposed on one side		Wall exposed on two sides		Wall exposed on one side		Wall exposed on two sides	
	a_s	t_w	a_s	t_w	a_s	t_w	a_s	t_w
min								
30	10	100	10	120	10	120	10	120
60	10	110	10	120	10	130	10	140
90	20	120	10	140	25	140	25	170
120	25	150	25	160	35	160	35	220
180	40	180	45	200	50	210	55	270
240	55	230	55	250	60	270	60	350

LEGEND:

a_s = axis distance

t_w = wall thickness

NOTES:

1 For $N_f^* / \phi N_u$ see Clause 5.6.3.

2 For prestressing tendons, the axis shall be increased, as specified in Clause 5.3.3.

5.7.3 Effective height limitations for walls

For walls required to have a FRL, the ratio of the effective height to thickness shall not exceed 40, where the effective height is determined from Clause 11.4. This latter restriction shall not apply to walls where the lateral support at the top of the wall is provided by an element not required by the relevant authority to have a FRL.

5.7.4 Other requirements for walls

5.7.4.1 Recesses for services in walls

The effect of recesses for services on the FRP for structural adequacy, integrity and insulation of a wall shall be ignored if the thickness of wall remaining under the bottom of the recess is not less than half the wall thickness and the total recessed area, within any 5 m² of wall face, is not more than 10 000 mm² on one or both faces of the wall.

If the above limits are exceeded, the wall thickness (t_w) used to determine FRP shall be taken as the overall thickness less the depth of the deepest recess.

5.7.4.2 *Effect of chases on structural adequacy of walls*

The effect of chases on the FRP for structural adequacy of walls shall be taken into account as follows:

- (a) For walls spanning one way, where—
 - (i) the chase direction is parallel to the span direction—ignored;
 - (ii) the chase direction is perpendicular to the span direction and of a length not greater than four times the wall thickness or 0.4 times the overall length of the wall, whichever is greater—ignored; or
 - (iii) the chase is perpendicular to the span direction and of a length greater than four times the wall thickness or 0.4 times the overall length of the wall—accounted for by using a slenderness ratio for the wall based on the reduced wall thickness.
- (b) For walls spanning two ways (panel action), where—
 - (i) there is either a vertical chase with a length not greater than half the wall height (H_w), or a horizontal chase with a length not greater than half the wall length (L_w)—ignored; and
 - (ii) the length of a vertical chase is greater than half the wall height (H_w), or the length of a horizontal chase is greater than half the wall length (L_w)—accounted for by using a slenderness ratio for the wall based on the reduced wall thickness, or the chase may be regarded as an unsupported edge and the panel designed as two sub-panels.

5.7.4.3 *Effect of chases on integrity and insulation of walls*

The effect of chases on the FRP for integrity and insulation of walls shall be taken into account as follows:

- (a) Where—
 - (i) the depth of the chase is not greater than 30 mm; and
 - (ii) the cross-sectional area of the chase, on a plane perpendicular to the plane of the wall face and at right angles to the centre-line of the chase, is not greater than 1000 mm²; and
 - (iii) the total face area of chases within any 5 m² of wall face is not greater than 100 000 mm² on one or both wall faces,
 the effect shall be ignored.
- (b) For cases other than those in Item (a) above, the effects shall be taken into account in accordance with the normal rules for insulation and integrity of walls, except that slenderness ratios shall be based on the reduced wall thickness.

5.8 INCREASE OF FIRE RESISTANCE PERIODS (FRPs) BY USE OF INSULATING MATERIALS

5.8.1 Increase of FRP by the addition of insulating materials

5.8.1.1 *General*

The FRP for insulation and structural adequacy of a concrete member may be increased by the addition to the surface of an insulating material, to provide increased thickness to the member, or greater insulation to the longitudinal reinforcement or tendons, or both.

5.8.1.2 *Acceptable forms of insulation*

Acceptable forms of insulation include the following:

- (a) Slabs of one part cement to four parts vermiculite (by volume) concrete or of one part cement to four parts perlite (by volume) concrete, bonded to the concrete.
- (b) Gypsum-vermiculite plaster or gypsum-perlite plaster, both mixed in the proportion of 0.16 m³ of aggregate to 100 kg of gypsum, in the form of either slabs bonded to the concrete, or as a sprayed or trowelled application applied in situ.
- (c) Any other fire-protective building board or material, which has been demonstrated to be equivalent for purpose in a standard fire-resistance test.

5.8.1.3 *Thickness of insulating material*

The minimum thickness of insulating material added to attain the required FRL shall be determined by testing in accordance with AS 1530.4.

In the absence of such testing and only for the materials specified in Clause 5.8.1.2, the minimum thickness of insulating material to be added may be taken as the difference between the required cover or effective thickness specified in this section and the actual cover or effective thickness, whichever governs, multiplied by—

- (a) 0.75, for materials specified in Clauses 5.8.1.2(a) and (b); or
- (b) a factor for materials as specified in Clause 5.8.1.2(c), where the factor is derived from tests in which the difference calculated above lies within the range of insulation thickness tested; and
- (c) the thickness thus calculated rounded to the nearest 5 mm above.

5.8.1.4 *Reinforcement in sprayed or trowelled insulating materials*

Where the thickness of sprayed or trowelled insulating materials exceeds 10 mm, it shall be reinforced to prevent detachment during exposure to fire.

5.8.2 **Increase of insulation period of slabs by application of toppings**

The FRP for insulation of a slab may be increased by incorporating an integral or a separately applied topping of thickness given by the following equation:

$$t_{\text{nom}} = kt_d + 10 \quad \dots 5.8.2$$

where

t_{nom} = nominal thickness of topping applied

k = 1.0 for a topping of plain concrete

= 0.8 for a topping of concrete made from lightweight aggregate conforming with AS 2758.1

= 0.6 for a topping of gypsum (including jointed gypsum block) having a wearing overlay

t_d = difference between the actual effective thickness of the slab and the effective thickness specified in Table 5.5.1, for the required FRP

SECTION 6 METHODS OF STRUCTURAL ANALYSIS

6.1 GENERAL

6.1.1 Basis for structural analysis

Methods of analysis for concrete structures shall take into account the following:

- (a) The strength and deformational properties of the member materials.
- (b) The equilibrium requirements for all forces acting on and within the structure.
- (c) The requirements of compatibility of deformations within the structure.
- (d) The support conditions and, where required, interaction of the structure with the foundation and other connecting or adjacent structures.

6.1.2 Interpretation of the results of analysis

Irrespective of the method chosen for the structural analysis, the simplifications, idealizations and assumptions implied in the analysis shall be considered in relation to the real, three-dimensional nature of the structure when the results of the analysis are interpreted.

NOTE: Users of software packages for analysis should ensure the package is appropriate for the analysis being undertaken.

6.1.3 Methods of analysis

For the purpose of conforming with the requirements for strength, serviceability and robustness specified in Section 2, it shall be permissible to determine the action effects and deformations in a reinforced or prestressed structure and its component members using one or more of the following methods:

- (a) Static analysis for determinate structures.
- (b) Linear elastic analysis, in accordance with Clause 6.2.
- (c) Linear elastic frame analysis incorporating secondary bending moments due to lateral joint displacement, in accordance with Clause 6.3.
- (d) Linear elastic stress analysis of members and structures, in accordance with Clause 6.4.
- (e) Non-linear frame analysis, in accordance with Clause 6.5.
- (f) Non-linear stress analysis of members and structures, in accordance with Clause 6.6.
- (g) Plastic methods of analysis for slabs and frames, in accordance with Clause 6.7.
- (h) Strut-and-tie method of analysis, in accordance with Clause 6.8.
- (i) Structural model tests designed and evaluated in accordance with the principles of mechanics.
- (j) The following simplified methods of analysis:
 - (i) The idealized frame method given in Clause 6.9.
 - (ii) The simplified methods given in Clause 6.10.

NOTE: Clause 2.2 allows different strength check procedures and different methods of analysis to be used for different members in a structure and for the structure.

6.1.4 Geometrical properties

For the purpose of this section, the definitions below apply.

6.1.4.1 Column strip

That portion of the design strip extending transversely from the centre-line of the supports—

- (a) for an interior column strip, one quarter of the distance to the centre-line of each adjacent and parallel row of supports; or
- (b) for an edge column strip, to the edge of the slab and one quarter of the distance to the centre-line of the next interior and parallel row of supports,

but of total width not greater than $L/2$ [see Figure 6.1.4(A)].

6.1.4.2 Design strip

That part of a two-way slab system, which is supported, in the direction of bending being considered, by a single row of supports and which in each span extends transversely from the centre-line of the supports—

- (a) for an interior design strip, halfway to the centre-line of each adjacent and parallel row of supports; or
- (b) for an edge design strip, to the edge of the slab and halfway to the centre-line of the next interior and parallel row of supports [see Figure 6.1.4(A)].

6.1.4.3 Middle strip

The portion of the slab between two column strips or between a column strip and a parallel supporting wall [see Figure 6.1.4(A)].

6.1.4.4 Span support

The length of a support in the direction of the span (a_{sup}) taken as—

- (a) for beams or for flat slabs without either drop panels or column capitals, the distance from the centre-line of the support to the face of the support; or
- (b) for flat slabs with drop panels or column capitals or both, the distance from the centre-line of the support to the intersection with the plane of the slab soffit of the longest line, inclined at an angle of 45° to the centre-line of the support, which lies entirely within the surfaces of the slab and the support, as shown in Figure 6.1.4(B).

NOTE: For the purpose of Item (b), circular or polygonal columns may be regarded as square columns with the same cross-sectional area.

6.1.4.5 Transverse width

The width of the design strip (L_t) measured perpendicular to the direction of bending being considered [see Figure 6.1.4(A)].

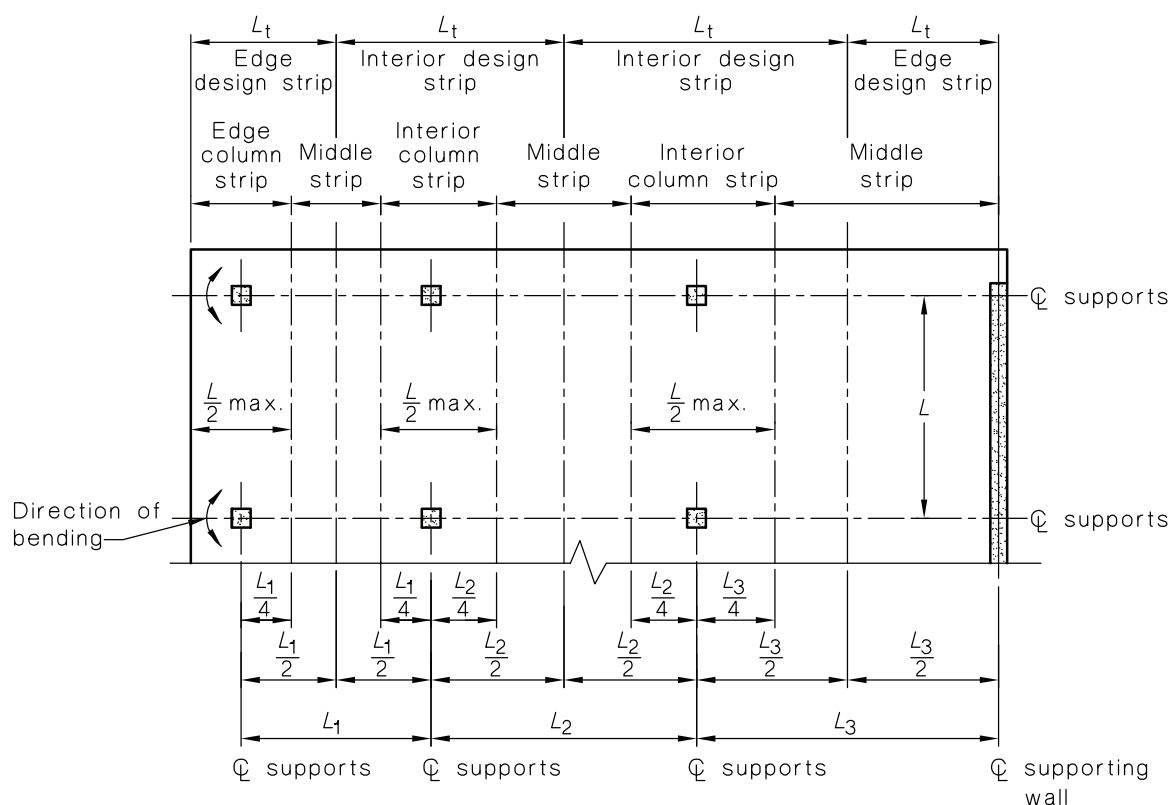


FIGURE 6.1.4(A) WIDTHS OF STRIPS FOR TWO-WAY SLAB SYSTEMS

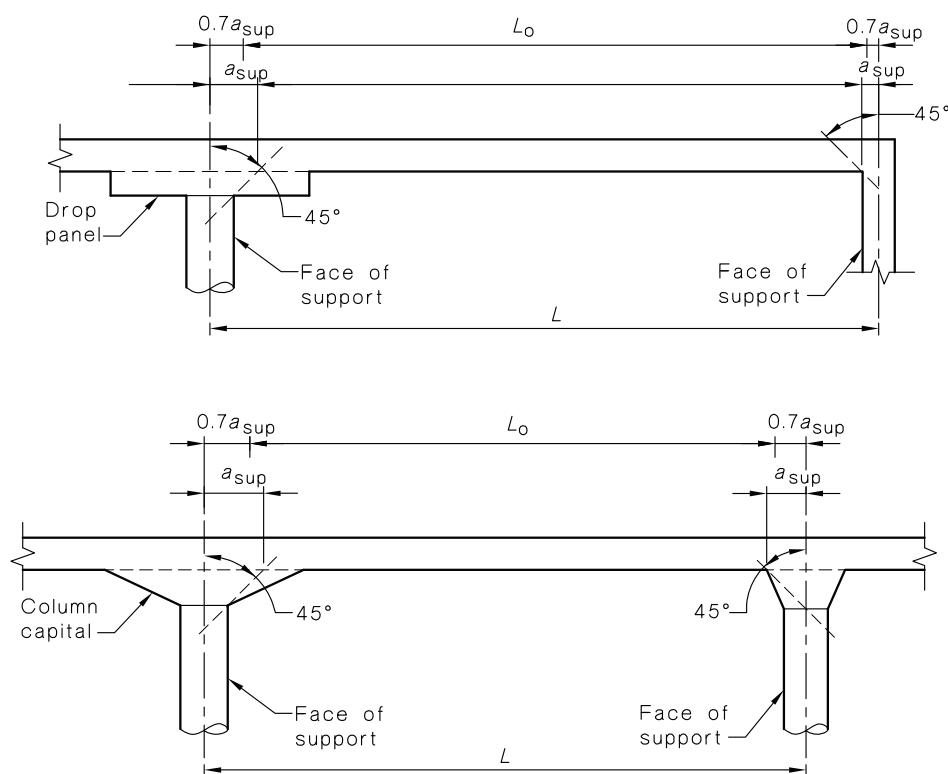


FIGURE 6.1.4(B) SPAN SUPPORT AND SPAN LENGTHS FOR FLAT SLABS

6.2 LINEAR ELASTIC ANALYSIS

6.2.1 General

This clause applies to the analysis of indeterminate continuous beams and framed structures in which secondary geometric effects are insignificant.

6.2.2 Span length

The span length of flexural members shall be taken as the distance centre-to-centre of supports.

6.2.3 Critical sections for negative moments

The critical section for a maximum negative bending moment shall be determined by analysis or may be taken at 0.7 times the length of a support in the direction of the span (a_{sup}) from the centre-line of the support.

6.2.4 Stiffness

6.2.4.1 General

The stiffness of members shall be chosen to represent the conditions at the limit state being analysed. The stiffness assumptions chosen shall be consistent with all loading conditions, and shall generate critical worst-case actions under all failure modes to be considered. Where multiple degrees of stiffness are possible, the stiffness assumption chosen for the failure mode under consideration shall induce worst-case actions in the element being designed.

The effect of haunching and other variations of cross-section along the axis of a member shall be considered and where significant, taken into account in the determination of the member stiffness.

Any assumptions regarding the relative stiffness of members shall be applied consistently throughout the analysis.

6.2.4.2 Stiffness of lateral force resisting elements

In the absence of refined analysis conducted in accordance with Section 6, the assessment of inter-storey drift, periods of vibration and distribution of internal actions, the moment of inertia for flexural members, columns and walls shall be determined as follows:

- (a) For uncracked sections, the full gross uncracked cross section of the member.

or

- (b) For cracked sections, in accordance with Table 6.2.4.

A section shall be assumed to be cracked unless the tensile stresses due to combined flexure and axial load at the limit state under consideration are less than the mean characteristic flexural tensile strength of the concrete determined in accordance with Clause 3.1.1.3.

TABLE 6.2.4
EFFECTIVE SECTION PROPERTIES,
 I_{eff} , AS A PROPORTION OF I_g

Type of member	Ultimate limit state
Beams and slabs	$0.4I_g$
Columns	
$N^*/(A_g f'_c) \geq 0.5$	$0.8I_g$
$N^*/(A_g f'_c) = 0.2$	$0.5I_g$
$N^*/(A_g f'_c) = 0.0$	$0.3I_g$
Walls	
$N^*/(A_g f'_c) \geq 0.2$	$0.4I_g$
$N^*/(A_g f'_c) = 0.1$	$0.3I_g$
$N^*/(A_g f'_c) = 0.0$	$0.25I_g$

NOTE: For intermediate values of $N^*/(A_g f'_c)$, linear interpolation is permitted.

6.2.5 Deflections

Deflection calculations shall take into account the effects of cracking, tension stiffening, shrinkage, creep, and relaxation of tendons. Calculations in accordance with the requirements of Clauses 8.5 and 9.3 shall be deemed to satisfy this requirement. Consideration shall be given to deformations that may result from deflection of the formwork or settlement of the supporting props during construction.

6.2.6 Secondary bending moments and shears resulting from prestress

The secondary bending moments and shears and the associated deformations that are produced in an indeterminate structure by prestressing shall be taken into account in the design calculations for serviceability.

NOTE: The secondary bending moments and shears due to the effects of prestress may be determined by elastic analysis of the unloaded uncracked structure.

In design calculations for strength, the secondary bending moments and shears due to prestress shall be included with a load factor of 1.0 when the design moments and shears for the load combinations given in Clause 2.5 are calculated. For the special case of permanent action (dead load) plus prestress at transfer, the load factors given by Clause 2.5 shall apply.

6.2.7 Moment redistribution in reinforced and prestressed members for strength design

6.2.7.1 General requirements

In design calculations for strength of statically indeterminate members, the elastically determined bending moments at any interior support may be reduced or increased by redistribution, provided an analysis is undertaken to show there is sufficient rotation capacity in critical moment regions to allow the assumed distribution of bending moments to be achieved.

The analysis shall take into account—

- the stress-strain curves of the steel reinforcement and tendons as defined in Clauses 3.2.3 and 3.3.3, respectively, assuming that fracture of the reinforcement and tendon occur at ϵ_{su} and ϵ_{pu} ;
- static equilibrium of the structure after redistribution of the moments;
- the properties of the concrete as defined in Clause 3.1; and

A2

- (d) stress-COD behaviour of SFRC.

6.2.7.2 Deemed-to-conform approach for reinforced and prestressed members

The requirement of Clause 6.2.7.1 shall be deemed to be met if the following requirements are satisfied:

- (a) All of the main reinforcement in the member shall be either Ductility Class N or E.
- (b) For SFRC members, the tensile reinforcement ratio for steel reinforcement and tendons is not less than 0.004.
- (c) The bending moment distribution before redistribution shall be determined in accordance with elastic analysis.
- (d) Where the neutral axis parameter (k_u) is less than or equal to 0.2 in all peak moment regions, the redistribution of the moment at a support shall not exceed 30%.
- (e) Where k_u exceeds 0.2 in one or more peak moment regions, but does not exceed 0.4, the redistribution shall not exceed $75(0.4 - k_u)\%$.
- (f) The positive bending moment shall be adjusted to maintain equilibrium.
- (g) Where k_u exceeds 0.4 in any peak moment region, no redistribution shall be made.
- (h) Static equilibrium of the structure after redistribution of the moments shall be used to evaluate all action effects for strength design.

NOTES:

- 1 The values of k_u are calculated for cross-sections that have been designed on the basis of the redistributed moment diagram.
- 2 The amount of redistribution is measured as a percentage of the bending moment before redistribution.
- 3 Extra checks should be made on ductility and the possibility of punching shear failures.

6.3 ELASTIC ANALYSIS OF FRAMES INCORPORATING SECONDARY BENDING MOMENTS

6.3.1 General

This clause applies to the elastic analysis of frames not restrained by bracing or shear walls, or both, for which the relative displacement at the ends of compressive members is less than $L_u/250$ under the design load for strength.

6.3.2 Analysis

An elastic analysis incorporating secondary bending moments shall conform with the requirements of Clause 6.2 and the following:

- (a) The effect of lateral joint displacements shall be taken into account.
- (b) For strength design of a regular rectangular framed structure, the cross-sectional stiffness of the flexural members and columns shall be determined in accordance with Clause 6.2.4.2.
- (c) For very slender members, the change in bending stiffness of a member due to axial compression shall be considered.

6.4 LINEAR ELASTIC STRESS ANALYSIS

6.4.1 General

This clause applies to the linear elastic stress analysis of structures and parts of structures using numerical methods, including finite element analysis.

6.4.2 Analysis

The analysis shall conform with the requirements of Clause 6.1.1. The results of the analysis shall be interpreted in accordance with the requirements of Clause 6.1.2.

6.4.3 Sensitivity of analysis to input data and modelling parameters

Checks shall be made to investigate the sensitivity of the results of a linear elastic stress analysis to variations in input data and modelling parameters.

6.5 NON-LINEAR FRAME ANALYSIS

6.5.1 General

This clause applies to the non-linear analysis of framed structures at service load, at overload, and at collapse. Non-linear analysis shall be carried out in accordance with the requirements of Clauses 6.1.1, 6.1.2 and 6.1.4.

6.5.2 Non-linear material effects

The analysis shall take into account all relevant non-linear and inelastic effects in the materials, such as—

- (a) non-linear relationship between stress and strain for the reinforcement, the tendons and the concrete;
- (b) cracking of the concrete;
- (c) the tension stiffening effect in the concrete between adjacent tensile cracks;
- (d) creep and shrinkage of the concrete; and
- (e) relaxation of tendons.

6.5.3 Non-linear geometric effects

Equilibrium of the structure in the deformed condition shall be considered whenever joint displacements or lateral deflections within the length of members affect the action effects or overall structural behaviour by more than 10 percent.

6.5.4 Values of material properties

When non-linear frame analysis is used as the basis for design, the calculations shall be undertaken using the mean values of all relevant material properties, such as concrete strength, initial elastic moduli and yield stress, and yield strain of the steel reinforcement and tendons.

Additional analysis shall be considered using other values of material properties to allow for variability of material properties and the effects of non-proportionality in non-linear analysis.

6.5.5 Sensitivity of analysis to input data and modelling parameters

Checks shall be made to investigate the sensitivity of the results of a non-linear frame analysis to variations in input data and modelling parameters.

6.6 NON-LINEAR STRESS ANALYSIS

6.6.1 General

This clause applies to the non-linear stress analysis of structures and parts of structures using numerical methods, including finite element analysis.

6.6.2 Analysis

The analysis shall conform with the requirements of Clause 6.1.1. The results of the analysis shall be interpreted in accordance with the requirements of Clause 6.1.2.

6.6.3 Non-linear material and geometric effects

The analysis shall take into account all relevant non-linear and inelastic effects, such as—

- (a) non-linear relation between stress and strain for the reinforcement, the tendons and the concrete;
- (b) cracking of the concrete;
- (c) the tension stiffening effect in the concrete between adjacent tensile cracks;
- (d) creep and shrinkage of the concrete;
- (e) relaxation of tendons; and
- (f) geometric non-linear effects.

6.6.4 Values of material properties

When non-linear stress analysis is used as the basis for design, the calculations shall be undertaken using the mean values of all relevant material properties chosen taking account of the effect of non-proportionality of the results, such as concrete strength, initial elastic moduli, and yield stress and yield strain of the steel reinforcement. Additional analysis shall be considered using other values of material properties to allow for variability.

6.6.5 Sensitivity of analysis to input data and modelling parameters

Checks shall be made to investigate the sensitivity of the results of a non-linear stress analysis to variations in input data and modelling parameters.

6.7 PLASTIC METHODS OF ANALYSIS

6.7.1 General

This clause applies to the plastic collapse analysis of frames, one-way and two-way slabs and slab-on-ground floors and pavements. The method may be used provided Ductility Class N reinforcement is used throughout for flexural reinforcement. For slab-on-ground floors and pavements containing steel fibres, the method may also be used provided that the requirements of Clause 16.3.3.8 are met.

Where plastic methods are used in the strength design of structures, the reinforcement shall be arranged with due regard to the serviceability requirements of the structure.

6.7.2 Methods for beams and frames

Plastic methods of analysis may be used for the strength design of continuous beams and frames in accordance with Clause 2.2.2, provided it is shown that the high-moment regions possess sufficient moment-rotation capacity to achieve the plastic redistribution determined in the analysis.

6.7.3 Methods for slabs

6.7.3.1 *Lower-bound method for slabs*

The design bending moments obtained using lower-bound theory shall satisfy the requirements of equilibrium and the boundary conditions applicable to the slab.

6.7.3.2 *Yield line method for slabs*

A yield line analysis for strength design of a slab shall satisfy the following requirements:

- (a) The design bending moments shall be obtained from calculations based on the need for a mechanism to form over the whole or part of the slab at collapse.
- (b) The mechanism that gives rise to the most severe design bending moments shall be used for the design of the slab.

6.8 ANALYSIS USING STRUT-AND-TIE MODELS

6.8.1 General

When strut-and-tie modelling is used in the analysis of a concrete structure or local region, the relevant requirements of Section 7 shall be satisfied.

6.8.2 Sensitivity of analysis to input data and modelling parameters

Checks shall be made to investigate the sensitivity of the results of a strut-and-tie analysis to variations in geometry and modelling parameters.

6.9 IDEALIZED FRAME METHOD OF ANALYSIS

6.9.1 General

This clause applies to the analysis of multistorey buildings of reinforced concrete and prestressed concrete that can be represented as a framework of line members with a regular layout. The clause also applies to the analysis of framed structures with a regular layout incorporating two-way slab systems as specified in Clause 6.9.5.

6.9.2 Idealized frames

The building framework shall be analysed by rigorous analysis or, alternatively, may be analysed as a series of idealized, approximately parallel, two-dimensional frames running in one main direction, and a second series of such frames running in the transverse direction.

Each idealized frame shall consist of the footings, the rows of vertical (or near-vertical) members and the horizontal (or near-horizontal) members they support at each floor level.

The analyses for vertical, horizontal and other loads shall be carried out for each idealized frame in accordance with Clause 6.2, 6.3 or 6.4 and the general requirements of Clauses 6.1.1 and 6.1.2.

The critical section for maximum negative bending moment in a floor of the idealized frame shall be determined by rigorous calculation or, alternatively, may be taken at 0.7 times the span support (a_{sup}) from the centre-line of the support.

6.9.3 Analysis for vertical loads

The arrangement of vertical loads to be considered in the analysis of an idealized frame shall be in accordance with Clause 2.5.4. In the analysis of a frame for vertical loads, the frame shall be analysed in its entirety. Alternatively, it shall be permissible to deal with one storey at a time, in accordance with the following:

- (a) To determine the moments and shears in a floor due to vertical loading, the floor together with the columns above and below shall be isolated and analysed, the columns being assumed fixed at the remote ends.

NOTE: The bending moment and shear at a given support may be determined on the assumption the floor is fixed at the support one span away, provided that the floor continues beyond that point.

- (b) To determine the forces and moments in columns due to vertical loading, each level of columns shall be considered together with the floors and columns above and below, the columns being assumed fixed against rotation and translation at their remote ends and the floors being assumed fixed at the adjacent supports.

Any change in length of the beams and slabs due to axial force and any deflection due to shear force may be neglected.

The effect of any change in length of columns, due to axial shortening on the actions in the floor system, shall be considered in the analysis.

In order to provide for imposed actions (live loads) acting on part of a span—

- (i) when the live load reduction factor $\psi_a = 1$, the minimum shear force due to imposed actions (live load) in any section of a member shall be taken as at least one quarter of the maximum shear force due to imposed actions (live load) in the member when subjected to uniformly distributed imposed actions (live loads); and
- (ii) when $\psi_a < 1$, the designer shall assess the maximum and minimum imposed actions (live loads) due to partial loading of a span.

6.9.4 Analysis for horizontal loads

In lieu of a rigorous analysis, in situ concrete floor slabs may be assumed to act as horizontal diaphragms that distribute lateral forces among the frames and walls. They shall be designed in accordance with Section 15.

The full idealized frame shall be considered in the analysis for horizontal loads, unless restraint is provided (for example by bracing or shear walls).

6.9.5 Idealized frame method for structures incorporating two-way slab systems

6.9.5.1 General

Provided Ductility Class L reinforcement is not used as the main flexural reinforcement, this clause applies to the analysis of reinforced and prestressed framed structures incorporating two-way slab systems having multiple spans including—

- (a) solid slabs with or without drop panels;
- (b) slabs incorporating ribs in two directions, including waffle-slabs;
- (c) slabs having recessed soffits, if the portion of reduced thickness lies entirely within both middle strips;
- (d) slabs having openings conforming with the requirements of Clause 6.9.5.5; and
- (e) beam-and-slab systems, including thickened slab bands.

6.9.5.2 Effective width

The idealized frame consists of the footings, the columns and the slab floors acting as wide beams.

The effective width of the beams to be used in the analysis varies depending on span length and column size, and may be different for vertical and lateral loads. In the absence of more accurate calculations, the stiffness of horizontal flexural members at each floor level for a vertical load analysis shall be based on a width—

- (a) for flat slabs, equal to the width of the design strip (L_1); or
- (b) for T-beams and L-beams, calculated in accordance with Clause 8.8.2.

6.9.5.3 Distribution of bending moments between column and middle strips

In the idealized frame each beam (design strip) shall be divided into column strips and middle strips.

The column strip shall be designed to resist the total negative or positive bending moment at the critical cross-sections multiplied by a factor within the ranges given in Table 6.9.5.3.

That part of the design strip bending moment not resisted by the column strip shall be proportionally assigned to the half-middle strips on either side of it.

Each middle strip shall be designed to resist the sum of the moments assigned to its two adjoining halves, except that a middle strip adjacent to and parallel with an edge supported by a wall shall be designed to resist twice the bending moment assigned to the adjoining half-middle strip from the next interior design strip parallel to the wall.

TABLE 6.9.5.3
DISTRIBUTION OF BENDING MOMENTS TO THE COLUMN STRIP

Bending moment under consideration	Strength limit state	Serviceability limit state
Negative moment at an interior support	0.60 to 1.00	0.75
Negative moment at an exterior support with spandrel beam	0.75 to 1.00	0.75
Negative moment at an exterior support without spandrel beam	0.75 to 1.00	1.0
Positive moment at all spans	0.50 to 0.70	0.6

6.9.5.4 Torsional moments

Where moment is transferred to the column by torsional moment in the slab or spandrel beams, the slab or spandrel beams shall be designed in accordance with Clause 9.2 or Clause 8.3, as applicable.

In beam-and-slab construction, the spandrel beams shall be reinforced with at least the minimum torsional reinforcement required by Clause 8.3.3.

6.9.5.5 Openings in slabs

Slabs containing openings may be analysed in accordance with all of Clause 6.9.5 without the need for further calculation provided the amount of reinforcement interrupted by the opening is distributed to each side of the opening and the plan dimensions of the opening are no larger than the following:

- (a) The width of each middle strip, in the area common to two middle strips.
- (b) One quarter of the width of each strip, in the area common to a column strip and a middle strip.
- (c) One eighth of the width of each column strip, in the area common to two column strips, provided the reduced section is capable of transferring the moment and shear forces to the support. The slab shall also conform with the shear requirements of Clause 9.2.

6.10 SIMPLIFIED METHODS OF FLEXURAL ANALYSIS

6.10.1 General

In lieu of more detailed structural analysis, it is permissible to design reinforced concrete beams and slabs for strength in accordance with the requirements of Clause 6.10.2, 6.10.3 or 6.10.4.

6.10.2 Simplified method for reinforced continuous beams and one-way slabs

6.10.2.1 Application

Clauses 6.10.2.2 to 6.10.2.4 may be used for the calculation of design bending moments and shear forces for strength in continuous beams and one-way slabs of reinforced concrete construction, provided—

- (a) the ratio of the longer to the shorter length of any two adjacent spans does not exceed 1.2;
- (b) the loads are essentially uniformly distributed;
- (c) the imposed action (live load) (q) does not exceed twice the permanent action (dead load) (g);
- (d) members are of uniform cross-section;

- (e) the reinforcement is arranged in accordance with the requirements of Clause 8.1.10.6 or Clause 9.1.3.2; and
- (f) bending moments at supports are caused only by the action of loads applied to the beam or slab.

6.10.2.2 Negative design moment

The negative design moment at the critical section, taken for the purpose of this clause at the face of the support, shall be as follows (where F_d is the uniformly distributed design load per unit length, factored for strength):

- (a) At the first interior support:
 - (i) Two spans only for Ductility Class N $F_d L_n^2 / 9$; or
for Ductility Class L $F_d L_n^2 / 8$.
 - (ii) More than two spans $F_d L_n^2 / 10$.
- (b) At other interior supports $F_d L_n^2 / 11$.
- (c) At interior faces of exterior supports for members built integrally with their supports:
 - (i) For beams where the support is a column $F_d L_n^2 / 16$.
 - (ii) For slabs and beams where the support is a beam $F_d L_n^2 / 24$.

6.10.2.3 Positive design moment

The positive design moment shall be taken as follows (where F_d is the uniformly distributed design load per unit length, factored for strength):

- (a) In an end span $F_d L_n^2 / 11$.
- (b) In interior spans for Ductility Class N $F_d L_n^2 / 16$; or
for Ductility Class L $F_d L_n^2 / 14$.

6.10.2.4 Transverse design shear force

The transverse design shear force in a member shall be taken as follows (where F_d is the uniformly distributed design load per unit length, factored for strength):

- (a) In an end span:
 - (i) At the face of the interior support $1.15 F_d L_n / 2$.
 - (ii) At midspan $F_d L_n / 7$.
 - (iii) At the face of the end support $F_d L_n / 2$.
- (b) In interior spans:
 - (i) At the face of supports $F_d L_n / 2$.
 - (ii) At midspan $F_d L_n / 8$.

6.10.3 Simplified method for reinforced two-way slabs supported on four sides

6.10.3.1 General

The design bending moments and shear forces for strength in reinforced two-way simply supported or continuous rectangular slabs, which are supported by walls or beams on four sides and have corners that are prevented from lifting, may be determined from Clauses 6.10.3.2 to 6.10.3.4 provided—

- (a) the loads are essentially uniformly distributed;
- (b) the reinforcement is arranged in accordance with the requirements of Clause 9.1.3.3;
- (c) bending moments at supports are caused only by the action of loads applied to the beam or slab;
- (d) any opening or penetration through the thickness of a slab does not adversely affect strength or stiffness; and
- (e) slabs incorporating Ductility Class L reinforcement are continuously supported on walls.

6.10.3.2 Design bending moments

The design bending moments in a slab shall be determined as follows:

- (a) The positive design bending moments at midspan, M_x^* and M_y^* on strips of unit width spanning L_x and L_y , respectively, shall be calculated from the following equations:

$$M_x^* = \beta_x F_d L_x^2 \quad \dots 6.10.3.2(1)$$

$$M_y^* = \beta_y F_d L_y^2 \quad \dots 6.10.3.2(2)$$

where F_d is the uniformly distributed design load per unit area factored for strength, and β_x and β_y are given in—

- (i) Table 6.10.3.2(A) for slabs with Ductility Class N reinforcement as the main flexural reinforcement; and
- (ii) Table 6.10.3.2(B) for slabs with Ductility Class N reinforcement or Ductility Class L mesh as the main flexural reinforcement, no moment redistribution can be accommodated at either the serviceability or strength limit states.

The moments, so calculated, shall apply over a central region of the slab equal to three-quarters of L_x and L_y respectively. Outside of this region, the requirement for strength shall be deemed to conform with the minimum strength requirement of Clause 9.1.1.

- (b) The negative design bending moments at a continuous edge shall be taken as—
 - (i) 1.33 times the midspan values in the direction considered, when they are taken from Table 6.10.3.2(A); or
 - (ii) α_x or α_y times the midspan values in the direction considered when they are taken from Table 6.10.3.2(B).

If the negative moment on one side of a common support is different from that on the other side—

- (A) the unbalanced moment may be redistributed (in proportion to the stiffness of span L_x in the adjacent panels) if Ductility Class N reinforcement is the main flexural reinforcement; or
- (B) the slab shall be reinforced on both sides of the support for the larger support moment.

- (c) The negative design bending moment at a discontinuous edge, where there is a likelihood of restraint, shall be taken as—
- (i) 0.5 times the midspan values in the direction considered when they are taken from Table 6.10.3.2(A); or
 - (ii) 0.8 times the midspan values in the direction considered when they are taken from Table 6.10.3.2(B).

TABLE 6.10.3.2(A)
BENDING MOMENT COEFFICIENTS FOR RECTANGULAR SLABS
SUPPORTED ON FOUR SIDES (DUCTILITY CLASS N REINFORCEMENT)

Edge condition		Short span coefficients (β_x)								Long span coefficients (β_y) for all values of L_y/L_x
		Values of L_y/L_x								
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	≥ 2.0	
1	Four edges continuous	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	0.024
2	One short edge discontinuous	0.028	0.032	0.036	0.038	0.041	0.043	0.047	0.050	0.028
3	One long edge discontinuous	0.028	0.035	0.041	0.046	0.050	0.054	0.061	0.066	0.028
4	Two short edges discontinuous	0.034	0.038	0.040	0.043	0.045	0.047	0.050	0.053	0.034
5	Two long edges discontinuous	0.034	0.046	0.056	0.065	0.072	0.078	0.091	0.100	0.034
6	Two adjacent edges discontinuous	0.035	0.041	0.046	0.051	0.055	0.058	0.065	0.070	0.035
7	Three edges discontinuous (one long edge continuous)	0.043	0.049	0.053	0.057	0.061	0.064	0.069	0.074	0.043
8	Three edges discontinuous (one short edge continuous)	0.043	0.054	0.064	0.072	0.078	0.084	0.096	0.105	0.043
9	Four edges discontinuous	0.056	0.066	0.074	0.081	0.087	0.093	0.103	0.111	0.056

TABLE 6.10.3.2(B)
BENDING MOMENT COEFFICIENTS FOR RECTANGULAR SLABS
SUPPORTED ON FOUR SIDES (DUCTILITY CLASS N
OR DUCTILITY CLASS L REINFORCEMENT)

Edge condition			Short span coefficients (β_x and α_x)									Long span coefficients (β_y and α_y) for all values of L_y/L_x
			Values of L_y/L_x									
			1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	>2.0	
1 Four edges continuous	β	0.021	0.025	0.029	0.032	0.034	0.036	0.039	0.041	0.042	0.020	
	α	2.31	2.22	2.14	2.10	2.06	2.03	2.00	2.00	2.00	2.69	
2 One short edge discontinuous	β	0.027	0.030	0.033	0.035	0.037	0.039	0.041	0.042	0.042	0.024	
	α	2.20	2.14	2.10	2.06	2.04	2.02	2.00	2.00	2.00	2.29	
3 One long edge discontinuous	β	0.024	0.028	0.034	0.038	0.043	0.047	0.056	0.061	0.070	0.028	
	α	2.22	2.17	2.09	2.03	1.97	1.93	1.86	1.81	1.80	2.46	
4 Two short edges discontinuous	β	0.032	0.035	0.037	0.038	0.039	0.040	0.042	0.042	0.042	0.024	
	α	2.09	2.05	2.03	2.01	2.00	2.00	2.00	2.00	2.00	—	
5 Two long edges discontinuous	β	0.024	0.028	0.035	0.042	0.049	0.056	0.071	0.085	0.125	0.039	
	α	—	—	—	—	—	—	—	—	—	2.31	
6 Two adjacent edges discontinuous	β	0.031	0.036	0.041	0.046	0.050	0.053	0.060	0.064	0.070	0.034	
	α	2.13	2.07	2.01	1.96	1.92	1.89	1.83	1.80	1.80	2.13	
7 Three edges discontinuous (one long edge continuous)	β	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.066	0.070	0.035	
	α	2.04	1.97	1.93	1.89	1.86	1.84	1.80	1.80	1.80	—	
8 Three edges discontinuous (one short edge continuous)	β	0.033	0.039	0.047	0.054	0.061	0.067	0.082	0.093	0.125	0.046	
	α	—	—	—	—	—	—	—	—	—	2.12	
9 Four edges discontinuous	β	0.044	0.052	0.059	0.066	0.073	0.079	0.091	0.100	0.125	0.049	
	α	—	—	—	—	—	—	—	—	—	—	

6.10.3.3 Torsional moment at exterior corners

The torsional moment at the exterior corners of a slab shall be deemed to be resisted by conforming with the requirements of Clause 9.1.3.3, Item (e).

6.10.3.4 Load allocation

For calculating shear forces in the slab or the forces applied to the supporting walls or beams in the absence of more accurate calculations, it may be assumed that the uniformly distributed load on the slab is allocated to the supporting beams or walls as shown in Figure 6.10.3.4, provided—

- (a) the reactions apply directly when all edges are continuous;
- (b) when one edge is discontinuous, the reactions on all continuous edges are increased by 10% and the reaction on the discontinuous edge can be reduced by 20%; and
- (c) when adjacent edges are discontinuous, the reactions are adjusted for elastic shear considering each span separately.

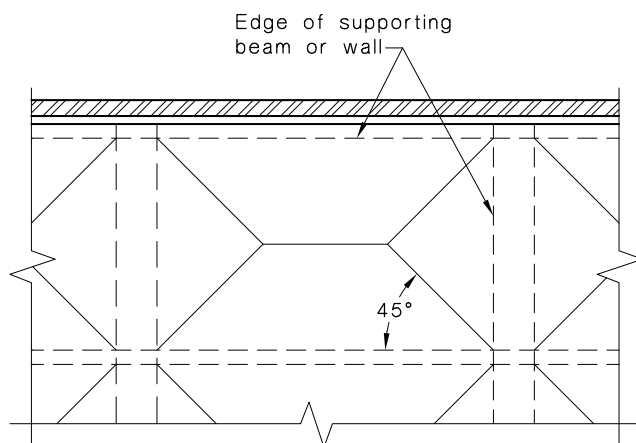


FIGURE 6.10.3.4 ALLOCATION OF LOAD

6.10.4 Simplified method for reinforced two-way slab systems having multiple spans

6.10.4.1 General

For multiple-span reinforced two-way slab systems, including solid slabs with or without drop panels, slabs incorporating ribs in two directions (waffle slabs) and beam-and-slab systems including thickened-slab bands, bending moments and shear forces in both directions may be determined in accordance with this clause provided—

- (a) there are at least two continuous spans in each direction;
- (b) the support grid is rectangular, except that individual supports may be offset up to a maximum of 10% of the span in the direction of the offset;
- (c) in any portion of the slab enclosed by the centre-lines of its supporting members, the ratio of the longer span to the shorter span is not greater than 2.0;
- (d) in the design strips in each direction, successive span lengths do not differ by more than one third of the longer span and in no case is an end-span longer than the adjacent interior span;
- (e) lateral forces on the structure are resisted by shear walls or braced frames;
- (f) vertical loads are essentially uniformly distributed;
- (g) the imposed action (live load) (q) does not exceed twice the permanent action (dead load) (g);
- (h) the reinforcement is arranged in accordance with Clause 9.1.3.4 or Clause 8.1.11.6, as applicable; and
- (i) Ductility Class L reinforcement is not used as the flexural reinforcement.

6.10.4.2 Total static moment for a span

The total static moment (M_o) for a span of the design strip shall be taken as not less than—

$$M_o = \frac{F_d L_t L_o^2}{8} \quad \dots 6.10.4.2$$

where

F_d = uniformly distributed design load per unit area, factored for strength

L_t = width of the design strip

$L_o = L$ minus 0.7 times the sum of the values of a_{sup} at each end of the span
[see Figure 6.1.4(B)]

6.10.4.3 Design moments

The design moments in a span shall be determined by multiplying the total static moment (M_o) by the relevant factor given in Tables 6.10.4.3(A) or 6.10.4.3(B).

These design moments may be modified by up to 10% provided the total static moment (M_o) for the span in the direction considered is not reduced.

The section under negative moment shall be designed to resist the larger of the two interior negative design moments determined for the spans framing into a common support, unless an analysis is made to distribute the unbalanced moment in accordance with the stiffness of the adjoining members.

TABLE 6.10.4.3(A)
DESIGN MOMENT FACTORS FOR AN END-SPAN

Type of slab system and edge rotation restraint	Exterior negative moment factor	Positive moment factor	Interior negative moment factor
Flat slabs with exterior edge unrestrained	0.0	0.60	0.80
Flat slabs with exterior edge restrained by columns only	0.25	0.50	0.75
Flat slabs with exterior edge restrained by spandrel beams and columns	0.30	0.50	0.70
Flat slabs with exterior edge fully restrained	0.65	0.35	0.65
Beam-and-slab construction	0.15	0.55	0.75

TABLE 6.10.4.3(B)
DESIGN MOMENT FACTORS FOR AN INTERIOR SPAN

Type of slab system	Negative moment factor	Positive moment factor
All types	0.65	0.35

6.10.4.4 Transverse distribution of the design bending moment

The design negative and positive bending moments shall be distributed to the column strip and middle strip in accordance with Clause 6.9.5.3.

6.10.4.5 Moment transfer for shear in flat slabs

For the purpose of shear design, the bending moment, transferred from the slab to the support (M_v^*), shall be taken as the unbalanced bending moment at that support.

At an interior support, M_v^* shall be taken as not less than—

$$0.06[(1.2g + 0.75q)L_t(L_o)^2 - 1.2gL_t(L_o')^2] \quad \dots 6.10.4.5$$

where

L_o' = smaller value of L_o for the adjoining spans

At an exterior support, the actual moment shall be taken.

6.10.4.6 *Shear forces in beam-and-slab construction*

In beam-and-slab construction, the shear forces in the supporting beams shall be determined by rigorous calculation or, alternatively, may be determined by using the allocation of load given in Clause 6.10.3.4.

6.10.4.7 *Openings in slabs*

Only openings that conform with the requirements of Clause 6.9.5.5(a) and Clause 6.9.5.5(b) shall be permitted in slabs.

SECTION 7 STRUT - AND - TIE MODELLING

7.1 GENERAL

It shall be permissible to use strut-and-tie models to represent the conditions at overload and at failure in non-flexural members and in non-flexural regions of members, as a basis both for strength design and for evaluating strength.

A strut-and-tie model shall consist of compression elements (struts) and tension elements (ties) that are connected together at nodes to form a load-resisting structural system.

Strut-and-tie models shall satisfy the following requirements:

- (a) Loads shall be applied at nodes, and the struts and ties shall be subjected only to axial force.
- (b) The model shall provide load paths to carry the loads and other actions to the supports or into adjacent regions.
- (c) The model shall be in equilibrium with the applied loads and the reactions.
- (d) In determining the geometry of the model, the dimensions of the struts, ties, and nodal zones shall be taken into account.
- (e) Ties shall be permitted to cross struts.
- (f) Struts shall cross or intersect only at nodes.
- (g) For reinforced concrete members at a node point, the angle between the axes of any strut and any tie shall be not less than 30°.
- (h) For prestressed concrete members at a node point, the angle between the axes of any strut and any tie with a tendon acting as the reinforcement shall be not less than 20°.

7.2 CONCRETE STRUTS

7.2.1 Types of struts

Struts shall be of prismatic, fan or bottle shape, depending on the geometry of the compression field, as shown in Figure 7.2.1. Prismatic struts shall be used only where the compressive stress field cannot diverge.

7.2.2 Strut efficiency factor

For prismatic struts, the strut efficiency factor (β_s) that is used to determine the design strength shall be taken as 1.0.

For fan-shaped and bottle-shaped compression fields that are unconfined, the strut efficiency factor shall be taken as—

$$\beta_s = \frac{1}{1.0 + 0.66 \cot^2 \theta} \quad (\text{within the limits } 0.3 \leq \beta_s \leq 1.0) \quad \dots 7.2.2$$

The angle (θ) is measured between the axis of the strut and the axis of a tie passing through a common node (see Figure 7.2.2). Where more than one tie passes through a node, or where the angle (θ) is different for nodes at each end of a strut, the smallest value of θ shall be used in determining β_s .

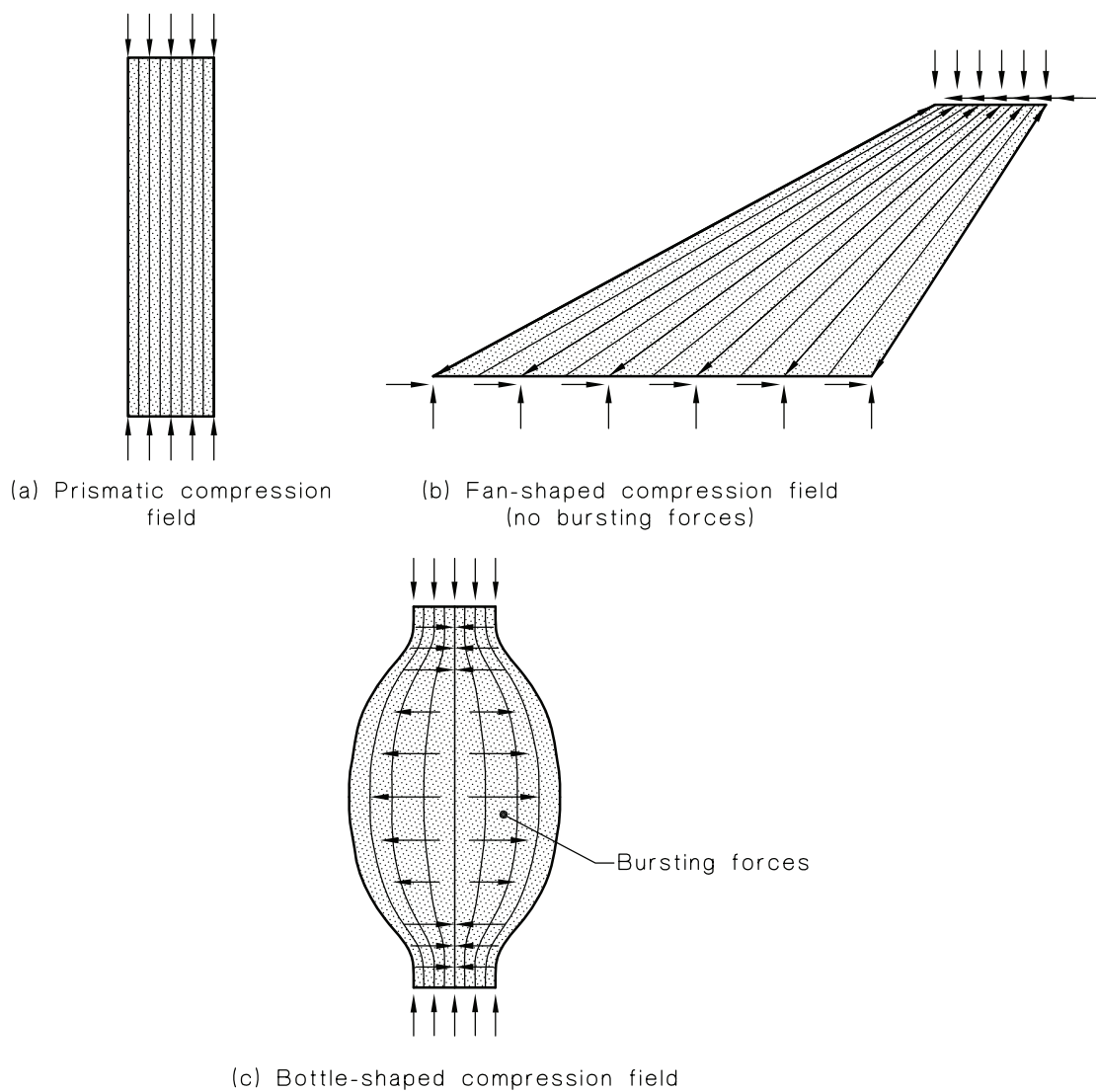
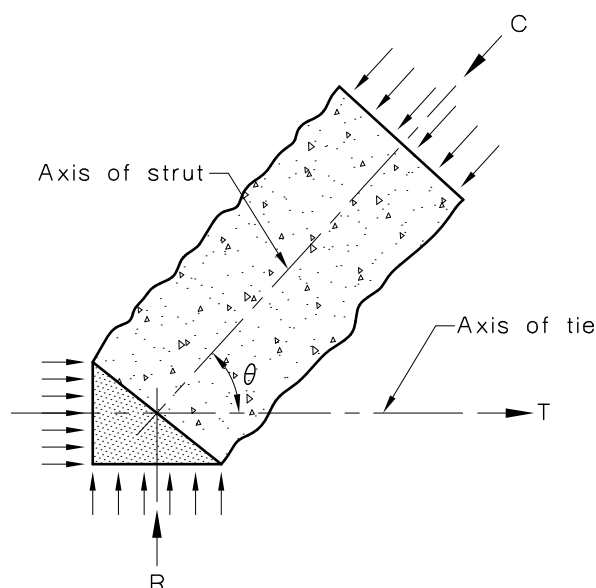


FIGURE 7.2.1 TYPES OF STRUTS

FIGURE 7.2.2 DEFINITION OF θ

7.2.3 Design strength of struts

The design strength of a concrete strut shall be taken as—

$$\phi_{st} \beta_s 0.9 f'_c A_c \quad \dots 7.2.3$$

where

A_c = smallest cross-sectional area of the concrete strut at any point along its length and measured normal to the line of action of the strut

β_s = an efficiency factor given in Clause 7.2.2

The value of the strength reduction factor (ϕ_{st}) shall be obtained from Table 2.2.4.

Where used to increase the strength of a strut, longitudinal reinforcement shall be—

- placed parallel to the axis of the strut;
- located within the strut;
- enclosed in ties or spirals satisfying Clause 10.7; and
- properly anchored.

The strength of a longitudinally reinforced strut shall be calculated as for a prismatic, pin-ended short column of similar geometry.

7.2.4 Bursting reinforcement in bottle-shaped struts

The design bursting force at both the serviceability limit state ($T_{b,s}^*$) and ultimate limit state (T_b^*) shall be calculated using an equilibrium model consistent with the bottle shape shown in Figure 7.2.4(A). The divergence angle (α) for the bottle-shaped strut shall be assessed for each situation but shall be not less than—

- $\tan \alpha = 1/2$ for serviceability; and
- $\tan \alpha = 1/5$ for strength.

The bursting force across the strut at cracking shall be taken as—

$$T_{b,cr} = 0.7 b l_b f'_{ct} \quad \dots 7.2.4(1)$$

where

b = width of rectangular cross section or member

l_b = length of the bursting zone [see Figure 7.2.4(A)]

If the calculated bursting force (T_b^*) is greater than $0.5T_{b,cr}$, with $\tan \alpha$ taken as $\frac{1}{2}$, then transverse reinforcement shall be provided in either—

- (i) two orthogonal directions at angles γ_1 and γ_2 to the axis of the strut [see Figure 7.2.4(B)]; or
- (ii) one direction at an angle γ_1 to the axis of the strut, where γ_1 shall be not less than 40° and shall satisfy the following:

(A) For serviceability

$$\Sigma A_{si} f_{si} \sin \gamma_i \geq \max(T_{b,s}^*, T_{b,cr}^*) \quad \dots 7.2.4(2)$$

(B) For strength

$$\phi_{st} \Sigma A_{si} f_{sy} \sin \gamma_i \geq T_b^* \quad \dots 7.2.4(3)$$

In the above expressions, A_{si} is the area of reinforcement in directions 1 and 2 crossing a strut at an angle γ_i to the axis of the strut [see Figure 7.2.4(B)] and f_{si} is the serviceability limit stress in the reinforcement as specified in Clause 12.7.

The transverse reinforcement shall be evenly distributed throughout the length of the bursting zone (l_b), which is given by—

$$l_b = \sqrt{z^2 + a^2} - d_c \quad \dots 7.2.4(4)$$

- A2 | and a , b and z are the shear span, the depth of the idealized strut, and the projection of the inclined compressive strut normal to the shear span respectively [see Figure 7.2.4(A)].



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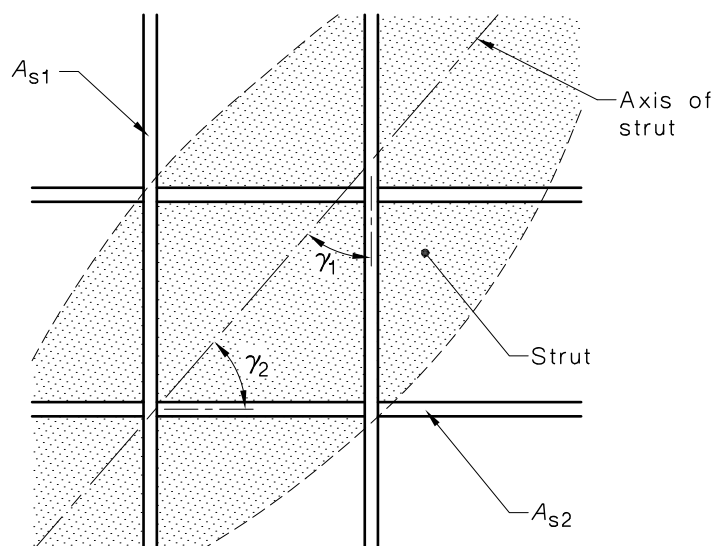


FIGURE 7.2.4(B) BURSTING REINFORCEMENT

7.3 TIES

7.3.1 Arrangement of ties

Ties shall consist of reinforcing steel and/or prestressing tendons. The reinforcement and/or tendons shall be evenly distributed across the nodal regions at each end of the tie, and arranged such that the resultant tensile force coincides with the axis of the tie in the strut-and-tie model.

7.3.2 Design strength of ties

The design strength of a tie shall be taken as $\phi_{st} [A_{st}f_{sy} + A_p(\sigma_{p,ef} + \Delta\sigma_p)]$ where $(\sigma_{p,ef} + \Delta\sigma_p)$ shall not exceed f_{py} . The value of ϕ_{st} shall be obtained from Table 2.2.4.

7.3.3 Anchorage of ties

To provide anchorage at each end of the tie, the reinforcement or tendon shall be extended beyond the node to achieve the design strength of the tie at the node and be anchored in accordance with Clause 13.1. At least 50% of the development length shall extend beyond the nodal zone.

Alternatively, anchorage of reinforcement may be achieved by a welded or mechanical anchorage, located entirely beyond the nodal zone.

7.4 NODES

7.4.1 Types of nodes

Three types of node are distinguished by the arrangement of the entering struts and ties, and the confinement thus provided, as follows:

- (a) CCC—there are only struts entering the node.
- (b) CCT—there are two or more struts and a single tension tie entering the node.
- (c) CTT—there are two or more tension ties entering the node.

7.4.2 Design strength of nodes

Where confinement is not provided to the nodal region, the design strength of the node shall be such that the principal compressive stress on any nodal face, determined from the normal and shear stresses on that face, is not greater than $\phi_{st}\beta_n 0.9f'_c$ where—

- (a) for CCC nodes $\beta_n = 1.0$; or
- (b) for CCT nodes $\beta_n = 0.8$; or
- (c) for CTT nodes $\beta_n = 0.6$.

The value of the strength reduction factor (ϕ_{st}) shall be taken from Table 2.2.4.

Where confinement is provided to the nodal region, the design strength of the node may be determined by tests or calculation, considering the confinement, but shall not exceed a value corresponding to a maximum compressive principal stress on any face of $\phi_{st}1.8f'_c$.

7.5 ANALYSIS OF STRUT-AND-TIE MODELS

In the analysis of a strut-and-tie model to determine the internal forces in the struts and ties, the requirements of Clause 6.1.1 shall be satisfied, and Clauses 6.1.2 and 6.8.2 shall be conformed with.

7.6 DESIGN BASED ON STRUT-AND-TIE MODELLING

7.6.1 Design for strength

When strut-and-tie modelling is used for strength design, the requirements of Clause 2.2.4 shall be satisfied.

7.6.2 Serviceability checks

When design for strength is based on strut-and-tie modelling, separate checks shall be undertaken to ensure that the design requirements for serviceability are satisfied.

SECTION 8 DESIGN OF BEAMS FOR STRENGTH AND SERVICEABILITY

8.1 STRENGTH OF BEAMS IN BENDING

8.1.1 General

The strength of a beam cross-section under bending shall be determined using Clauses 8.1.2 to 8.1.10 and the material properties given in Section 3. Beam properties for T-beams and L-beams shall be as given in Clause 8.8.

This clause does not apply to non-flexural members covered by Section 12.

8.1.2 Basis of strength calculations

Calculations for strength of cross-sections in bending shall incorporate equilibrium and strain-compatibility considerations and be consistent with the following assumptions:

- (a) Plane sections normal to the axis remain plane after bending, except for unbonded tendons (see Clause 8.1.8).
- (b) The concrete has no tensile strength.
- (c) The distribution of compressive stress is determined from a stress-strain relationship for the concrete in accordance with Clause 3.1.4 (see Note 1).
- (d) The strain in compressive reinforcement does not exceed 0.003.

NOTES:

- 1 If a curvilinear stress-strain relationship is used, then—
 - (a) Clause 3.1.4 places a limit on the value of the maximum concrete stress; and
 - (b) the strain in the extreme compression fibre may be adjusted to obtain the maximum bending strength.
- 2 These rules apply to reinforced and bonded prestressed concrete members.

8.1.3 Rectangular stress block

Clause 8.1.2 shall be deemed to be satisfied for the concrete assuming that—

- (a) the maximum strain in the extreme compression fibre is taken as 0.003; and
- (b) a uniform compressive stress of $\alpha_2 f'_c$ acts on an area bounded by—
 - (i) the edges of the cross-section; and
 - (ii) a line parallel to the neutral axis under the loading concerned, and located at a distance $\gamma k_d d$ from the extreme compressive fibre, where—

$$\alpha_2 = 0.85 - 0.0015 f'_c \quad \dots \alpha_2 \geq 0.67 \quad \dots 8.1.3(1)$$

$$\gamma = 0.97 - 0.0025 f'_c \quad \dots \gamma \geq 0.67 \quad \dots 8.1.3(2)$$

NOTES:

- 1 The modification of $0.9 f'_c$ given in Clause 3.1.4 is included in the rectangular stress block assumptions.
- 2 For circular sections α_2 shall be reduced by 5 percent and for any other sections where the width reduces from the neutral axis towards the compression face, α_2 shall be reduced by 10 percent.

A1

A2 | 8.1.4 Dispersion angle of prestress and concentrated forces

In the absence of a more exact calculation, the dispersion angle when a concentrated load is applied to a bearing surface shall be 60° (i.e. 30° either side of the line of action of the concentrated load).

NOTE: This Clause includes the dispersion angle of a prestressing force applied at an anchorage.

Tensile splitting or bursting forces that arise from dispersion of a concentrated force shall be designed in accordance with Section 7.

8.1.5 Design strength in bending

The design strength in bending of a section shall be taken as not greater than ϕM_{uo} , where ϕ is determined from Item (b) of Table 2.2.2.

A1 | Sections with k_{uo} greater than 0.36 and where $M^* > 0.8\phi M_{uo}$ shall be used only when—

(a) the structural analysis is carried out in accordance with Clauses 6.2 to 6.6; and

A2 | (b) compressive reinforcement of at least 0.01 times the area of concrete in compression is used and restrained as specified in Clauses 8.3.1.6 and 10.7.4, as appropriate.

NOTE: Where applicable the ductility requirements of Clause 14.4.6 shall be satisfied.

8.1.6 Minimum strength requirements

8.1.6.1 General

The ultimate strength in bending (M_{uo}) at critical sections shall be not less than $(M_{uo})_{min}$, the minimum required strength in bending at a critical cross-section, and is given by—

$$(M_{uo})_{min} = 1.2 [Z(f'_{ct,f} + P_e / A_g) + P_e e] \quad \dots 8.1.6.1(1)$$

where

Z = section modulus of the uncracked cross-section, referred to the extreme fibre at which flexural cracking occurs

$f'_{ct,f}$ = characteristic flexural tensile strength of the concrete

P_e = total effective prestress force allowing for all losses of prestress

e = eccentricity of the prestressing force (P_e), measured from the centroidal axis of the uncracked section

This requirement may be waived at critical sections of a statically indeterminate member, provided it can be demonstrated this will not lead to sudden collapse of a span or a reduced collapse load.

For reinforced concrete cross-sections, this requirement shall be deemed to be satisfied for the direction of bending being considered if tensile reinforcement (A_{st}) is provided such that—

$$A_{st} \geq [\alpha_b (D/d)^2 f'_{ct,f} / f_{sy}] b_w d \quad \dots 8.1.6.1(2)$$

where

For rectangular sections:

$$\alpha_b = 0.20$$

For T-sections and L-sections with the web in tension:

$$\alpha_b = 0.20 + \left(\frac{b_{ef}}{b_w} - 1 \right) \left(0.4 \frac{D_s}{D} - 0.18 \right) \geq 0.20 \left(\frac{b_{ef}}{b_w} \right)^{1/4}$$

For T-sections and L-sections with the flange in tension:

$$\alpha_b = 0.20 + \left(\frac{b_{ef}}{b_w} - 1 \right) \left(0.25 \frac{D_s}{D} - 0.08 \right) \geq 0.20 \left(\frac{b_{ef}}{b_w} \right)^{2/3}$$

8.1.6.2 Prestressed beams at transfer

The strength of a prestressed beam at transfer shall be checked using the load combinations specified in Clause 2.5.2.2 and a strength reduction factor (ϕ) for the section of 0.6.

This requirement shall be deemed to be satisfied if the maximum compressive stress in the concrete, under the design loads at transfer, does not exceed—

- (a) for a section that is rectangular in cross section and where the stress distribution is triangular in shape, $0.6 f'_{cp}$; otherwise
- (b) $0.5 f'_{cp}$ where f'_{cp} is the characteristic strength of concrete at transfer.

8.1.7 Stress in reinforcement and bonded tendons at ultimate strength

The stress in the reinforcement at ultimate strength shall be taken as not greater than f_{sy} .

In the absence of a more accurate calculation and provided the minimum effective stress in the tendons is not less than $0.5 f_{pb}$, the maximum stress that would be reached in bonded tendons at ultimate strength (σ_{pu}) shall be taken as—

$$\sigma_{pu} = f_{pb} \left(1 - \frac{k_1 k_2}{\gamma} \right) \quad \dots 8.1.7(1)$$

where

$k_1 = 0.4$ generally; or

if $f_{py}/f_{pb} \geq 0.9$,

$k_1 = 0.28$; and

$$k_2 = \frac{1}{b_{ef} d_p f'_c} [A_{pt} f_{pb} + (A_{st} - A_{sc}) f_{sy}]$$

Compressive reinforcement may be taken into account only if d_{sc} , the distance from the extreme compressive fibre of the concrete to the centroid of compressive reinforcement, is not greater than $0.15 d_p$, in which case k_2 shall be taken as not less than 0.17.

8.1.8 Stress in tendons not yet bonded

Where the tendon is not yet bonded, the stress in the tendon at ultimate strength (σ_{pu}) shall be determined from the formula given in Item (a) below if the span-to-depth ratio is 35 or less, or from the formula given in Item (b) below if the span-to-depth ratio is greater than 35, but in no case shall σ_{pu} be taken greater than f_{py} :

$$(a) \quad \sigma_{pu} = \sigma_{p,ef} + 70 + \frac{f'_c b_{ef} d_p}{100 A_{pt}} \leq \sigma_{p,ef} + 400 \quad \dots 8.1.8(1)$$

$$(b) \quad \sigma_{pu} = \sigma_{p,ef} + 70 + \frac{f'_c b_{ef} d_p}{300 A_{pt}} \leq \sigma_{p,ef} + 200 \quad \dots 8.1.8(2)$$

where $\sigma_{p,ef}$ is the effective stress in the tendon after allowing for all losses.

8.1.9 Spacing of reinforcement and tendons

The minimum clear distance between parallel bars (including bundles of bars), ducts and tendons shall be such that the concrete can be properly placed and compacted in accordance

with Clause 17.1.3. The maximum spacing of longitudinal reinforcement and tendons shall be determined in accordance with Clause 8.6.1(b).

8.2 STRENGTH OF BEAMS IN SHEAR

8.2.1 General

8.2.1.1 Combined flexure, torsion and shear

Clause 8.2 applies to reinforced and prestressed beams subjected to any combination of torsion, flexure, and shear and axial loads. It does not apply to non-flexural members covered by Sections 7 and 12.

8.2.1.2 Consideration of torsion

Where torsional strength is not required for the equilibrium of the structure and the torsion in a member is induced solely by the angular rotation of adjoining members, the torsional stiffness in the analysis and torsion in the member may be disregarded, provided the minimum torsion reinforcement requirements of Clause 8.2.5.5 and the detailing requirements of Clause 8.3.3 are satisfied.

Torsional effects shall be considered in regions where—

$$T^* > 0.25\phi T_{cr} \quad \dots 8.2.1.2(1)$$

where

$$T_{cr} = 0.33\sqrt{f'_c} \frac{A_{cp}^2}{u_c} \sqrt{1 + \frac{\sigma_{cp}}{0.33\sqrt{f'_c}}} \quad \dots 8.2.1.2(2)$$

and

A_{cp} = total area enclosed by outside perimeter of concrete section

T_{cr} = torsional cracking moment

u_c = the length of the outside perimeter of concrete cross-section

σ_{cp} = average intensity of effective prestress in concrete at the centroid, or at the junction of the web and flange when the centroid lies inside the flange

T^* = torsional moment calculated on an uncracked sectional analysis

For box section structures the term A_{cp}^2/u_c in Equation 8.2.1.2(2) shall not exceed $2A_o b_v$

where

A_o = area enclosed by shear flow path, including any area of holes therein

b_v = effective width of the web (see Clause 8.2.1.5)

[Text deleted]

8.2.1.3 Vertical component of prestress

Where the factored vertical component of prestress $\gamma_p P_v$ is included in Clause 8.2, including its subclauses, and where—

(a) P_v acts to reduce the shear stress resultant, then P_v is positive and $\gamma_p = 0.90$; or

(b) P_v acts to increase the shear stress resultant, then P_v is negative and $\gamma_p = 1.15$.

8.2.1.4 Tapered members

In members that are tapered along their length, the components of inclined tensile or compressive forces shall be taken into consideration in the calculation of shear strength.

8.2.1.5 Effective web width

The effective web width (b_v) shall be taken as the minimum web width within the effective shear depth (d_v). In determining b_v at a particular level, for prestressing ducts with $\Sigma d_d \geq b_w/8$:

$$b_v = (b_w - k_d \Sigma d_d) \quad \dots 8.2.1.5$$

where

Σd_d = sum of the widths of prestressing ducts, if any, in a horizontal plane across the web

k_d = 0.5 for grouted steel duct

= 0.8 for grouted plastic duct

= 1.2 for ungrouted duct

NOTES:

1 k_d may be reduced in the presence of reinforcement transverse to the plane of the web.

2 For solid circular sections, b_w may be taken as the section diameter.

8.2.1.6 Requirements for transverse shear reinforcement

Transverse reinforcement shall be provided in all regions for any of the following cases:

(a) $V^* - \gamma_p P_v > k_s \phi V_{uc} \quad \dots 8.2.1.6(1)$

where

(i) $D \leq 300$ mm, $k_s = 1.0$; or

(ii) 300 mm $< D < 650$ mm, $k_s = (1000 - D)/700$; or

(iii) $D \geq 650$ mm, $k_s = 0.5$

and γ_p is given in Clause 8.2.1.3.

Irrespective of the above, where it is shown by rational calculation satisfying conditions of equilibrium and compatibility that a shear failure of the structural component under consideration will not cause collapse of the structure, k_s may be taken as 1.0.

(b) $T^* > 0.25 \phi T_{cr} \quad \dots 8.2.1.6(2)$

(c) The overall depth of the beam or one-way slab is $D \geq 750$ mm $\dots 8.2.1.6(3)$

8.2.1.7 Minimum transverse shear reinforcement

Where transverse reinforcement is required, the minimum area ($A_{sv,min}$) provided in a concrete beam or slab shall be calculated from the following equation:

$$\frac{A_{sv,min}}{s} = \frac{0.08 \sqrt{f'_c} b_v}{f_{sy,f}} \quad \dots 8.2.1.7$$

where

s = centre-to-centre spacing of shear reinforcement, measured parallel to the longitudinal axis of the member

8.2.1.8 Design yield strength of tendons as transverse shear reinforcement

The design yield strength of tendons used as transverse shear reinforcement shall be taken as the effective prestress plus 500 MPa, but shall not be taken greater than f_{py} .

8.2.1.9 Effective shear depth

The effective shear depth (d_v) shall be taken as the greater of $0.72D$ or $0.9d$, where d is the distance from the extreme compression fibre to the centroid of the longitudinal tension reinforcement in the half-depth of the section containing the flexural tension zone.

8.2.2 Design procedures

8.2.2.1 Flexural regions

Where it is reasonable to assume that plane sections remain plane after loading, regions of members shall be designed for shear and torsion using either the sectional model in Clause 8.2.3 or by the strut and tie model in Section 7. The requirements of Clause 8.2.1 shall apply.

8.2.2.2 Regions near discontinuities

Where the plane sections assumption of flexural theory is not valid, regions of members shall be designed for shear and torsion using the strut-and-tie model as specified in Section 7, and Clause 12.2 shall apply.

8.2.2.3 Interface regions

Interfaces between elements shall be designed for shear transfer in accordance with Clause 8.4.

8.2.2.4 Detailed analysis

In lieu of the methods specified in Clauses 8.2.2.1 to 8.2.2.3, the resistance of members in shear or in shear combined with torsion may be determined by satisfying the applicable conditions of equilibrium and compatibility of strains, using appropriate stress-strain relationships for reinforcement and for diagonally cracked concrete.

8.2.3 Sectional design of a beam

8.2.3.1 Design strength of a beam in shear and torsion

The design shear strength of a beam (ϕV_u) shall satisfy the following:

$$\phi V_u \geq V^* - \gamma_p P_v \quad \dots 8.2.3.1(1)$$

where

$$V_u = V_{uc} + V_{us}$$

and

$$V_{uc} = \text{determined from Clause 8.2.4}$$

$$V_{us} = \text{determined from Clause 8.2.5}$$

$$P_v = \text{vertical component of prestress at the section under consideration (see Clause 8.2.1.3)}$$

$$\gamma_p = \text{prestress load factor determined in accordance with Clause 8.2.1.3}$$

The design torsional strength of a beam (ϕT_{us}) shall satisfy:

$$\phi T_{us} \geq T^* \quad \dots 8.2.3.1(2)$$

In addition, web crushing provisions of Clause 8.2.3.3 shall be satisfied.

NOTES:

- 1 Values for ϕ are given in Table 2.2.2.
- 2 Transverse reinforcement calculated for shear and for torsion are additive (see Clause 8.2.5.3).

A2

8.2.3.2 Maximum transverse shear near a support

The maximum transverse shear near a support shall be taken as the shear at—

- (a) the face of the support; or
- (b) a distance of d_v from the face of the support, provided—
 - (i) the member is directly supported and diagonal cracking cannot take place at the support or extend into it; and
 - (ii) the transverse shear reinforcement required at d_v from the support is continued unchanged to the face of the support.

Members where the distance from the point of zero shear to the face of the support is less than $2d_v$, or members in which a concentrated load is closer than $2d_v$ from the face of the support and contributes more than 50% of the design shear at a support, may be considered to be deep components for which the provisions of Section 12 apply.

For concentrated loads near a support, either the maximum transverse shear shall be taken at the face of the support or refined modelling techniques used to consider the enhanced effect of loads taken directly to a support, such as strut-and-tie modelling.

Notwithstanding the above, the maximum transverse shear at the face of the support shall satisfy the requirements of Clause 8.2.3.3.

8.2.3.3 Shear and torsional strength limited by web crushing

The combined shear and torsional strength at a cross-section is limited by the shear force when web crushing occurs ($V_{u,max}$), given by—

$$V_{u,max} = 0.55 \left[0.9 f'_c b_v d_v \left(\frac{\cot \theta_v + \cot \alpha_v}{1 + \cot^2 \theta_v} \right) \right]; \text{ or} \quad \dots 8.2.3.3(1)$$

$$V_{u,max} = 0.55 \left[0.9 f'_{cp} b_v d_v \left(\frac{\cot \theta_v + \cot \alpha_v}{1 + \cot^2 \theta_v} \right) \right] \text{ at transfer} \quad \dots 8.2.3.3(2)$$

where

d_v = effective shear depth (see Clause 8.2.1.9)

θ_v = angle of inclination of the compressive strut (see Clause 8.2.4.2)

α_v = angle between the transverse reinforcement and the longitudinal tensile reinforcement

To avoid web crushing failure, the following shall be satisfied:

- (a) For box sections:

- (i) Where wall thickness $t_w > A_{oh}/u_h$

$$\frac{V^* - \gamma_p P_v}{b_v d_v} + \frac{T^* u_h}{1.7 A_{oh}^2} \leq \frac{\phi V_{u,max}}{b_v d_v} \quad \dots 8.2.3.3(3)$$

- (ii) Where wall thickness $t_w \leq A_{oh}/u_h$

$$\frac{V^* - \gamma_p P_v}{b_v d_v} + \frac{T^*}{1.7 t_w A_{oh}} \leq \frac{\phi V_{u,max}}{b_v d_v} \quad \dots 8.2.3.3(4)$$

(b) For other sections:

A2

$$\sqrt{\left[\frac{V^*}{b_v d_v}\right]^2 + \left[\frac{T^* u_h}{1.7 A_{oh}^2}\right]^2} \leq \frac{\phi V_{u,max}}{b_v d_v} \quad \dots 8.2.3.3(5)$$

where

A_{oh} = area enclosed by centre-line of exterior closed transverse torsion reinforcement, including area of holes (if any)

u_h = perimeter of the centre-line of the closed transverse torsion reinforcement

$V_{u,max}$ = ultimate shear strength limited by web crushing failure

t_w = minimum wall thickness of the outer webs

8.2.4 Concrete contribution to shear strength (V_{uc})

8.2.4.1 General

The shear strength may be determined by any of the following, provided all requirements for use of the chosen model are satisfied—

$$V_{uc} = k_v b_v d_v \sqrt{f'_c} \quad \dots 8.2.4.1$$

A2

where $\sqrt{f'_c}$ shall not exceed 8.0 MPa. The value of k_v and the concrete strut angle (θ_v) shall be determined in accordance with the general method of Clause 8.2.4.2. Alternatively, for non-prestressed components not subject to axial tension or torsion, and provided the specified yield strength of the longitudinal reinforcement does not exceed 500 MPa, the design concrete strength does not exceed 65 MPa and the size of maximum aggregate particle is not less than 10 mm the value of k_v and the angle of the strut θ_v may be determined by the simplified method of Clause 8.2.4.3.

8.2.4.2 Determination of k_v and θ_v (general method)

8.2.4.2.1 Equations

A2

The angle of inclination of the concrete compression strut to the longitudinal axis of the member (θ_v) shall be calculated as follows:

$$\theta_v = (29 + 7000 \varepsilon_x) \quad \dots 8.2.4.2(1)$$

The value of k_v shall be calculated as follows:

(a) For $A_{sv}/S < A_{sv,min}/S$:

A2

$$k_v = \left[\frac{0.4}{1 + 1500 \varepsilon_x} \right] \left[\frac{1300}{1000 + k_{dg} d_v} \right] \quad \dots 8.2.4.2(2)$$

where

(i) $f'_c \leq 65$ MPa and not light-weight concrete

A2

$$k_{dg} = \left[\frac{32}{(16 + d_g)} \right] \text{ but not less than } 0.8 \quad \dots 8.2.4.2(3)$$

d_g = maximum nominal aggregate size

Provided the maximum nominal aggregate size (d_g) is not less than 16 mm, k_{dg} may be taken as 1.0.

(ii) $f'_c > 65$ MPa or light-weight concrete

A2	$k_{dg} = 2.0$... 8.2.4.2(4)
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A1	(b) For $A_{sv} / s \geq A_{sv,min} / s$:
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A2	$k_v = \left[\frac{0.4}{1 + 1500\varepsilon_x} \right]$... 8.2.4.2(5)
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where ε_x is the longitudinal strain in the concrete at the mid-depth of the section, determined in accordance with Clause 8.2.4.2.2.

8.2.4.2.2 Determination of the longitudinal strain in concrete ε_x for shear

In lieu of more accurate calculations, the longitudinal strain in the concrete at the mid-depth of the section (ε_x) shall be calculated as follows:

A2	$\varepsilon_x = \frac{ M^*/d_v + V^* - \gamma_p P_v + 0.5N^* - A_{pt}f_{po}}{2(E_s A_{st} + E_p A_{pt})} \leq 3.0 \times 10^{-3}$... 8.2.4.2.2(1)
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Where ε_x as calculated above is less than zero (i.e. compression), ε_x may either be taken as zero or calculated as follows:

A2	$\varepsilon_x = \frac{ M^*/d_v + V^* - \gamma_p P_v + 0.5N^* - A_{pt}f_{po}}{2(E_s A_{st} + E_p A_{pt} + E_c A_{ct})}$ within the limits $-0.2 \times 10^{-3} \leq \varepsilon_x \leq 0$... 8.2.4.2.2(2)
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where

- | | |
|----|---|
| A2 | <p>(a) M^* and V^* are absolute values and M^* shall not be taken as less than $(V^* - \gamma_p P_v) d_v$.</p> <p>(b) N^* is taken as positive for tension and negative for compression.</p> <p>NOTE: For rigid frames the value of N^* used to determine ε_x may be taken as twice the compressive axial thrust calculated by elastic analysis.</p> <p>(c) A_{st} and A_{pt} are the areas of reinforcing bars and prestressing tendons, respectively, located in the half depth of the section containing the flexural tensile zone (between $D/2$ and the extreme tensile fibre).</p> <p>(d) f_{po} is the stress in prestressing tendons when strain in the surrounding concrete is zero.</p> <p>(e) A_{ct} is the area of concrete between the mid depth of the section and the extreme tensile fibre.</p> |
|----|---|

A2	NOTE: f_{po} may be taken as $0.5f_{pb}$ for bonded tendons outside the transfer length and σ_p for unbonded tendons.
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In calculating A_{st} , the area of bars that terminate less than their development length from the section under consideration shall be reduced in proportion to their lack of full development.

A2	For sections closer than d_v to the face of the support, the value of ε_x calculated at d_v from the face of the support may be used in evaluating k_v and θ_v .
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If the axial tension is large enough to crack the flexural compression face of the section, the resulting increase in ε_x shall be considered. In lieu of more accurate calculations, ε_x calculated from the equation shall be doubled.

k_v and θ_v may be determined from Clause 8.2.4.2 using a value of ε_x that is greater than that calculated from the equation in this Clause. The mid-depth strain parameter ε_x shall not be greater than 3.0×10^{-3} .

8.2.4.2.3 Determination of ε_x for combined shear and torsion

In lieu of more accurate calculations, the longitudinal strain in the concrete at the mid-depth of the section (ε_x) subjected to shear and torsion shall be calculated as follows:

$$\varepsilon_x = \frac{\left| \frac{M^*}{d_v} \right| + \sqrt{\left(|V^*| - \gamma_p P_v \right)^2 + \left[\frac{0.9T^* u_h}{2A_o} \right]^2} + 0.5N^* - A_{pt}f_{po}}{2(E_s A_{st} + E_p A_{pt})} \leq 3.0 \times 10^{-3} \quad \dots 8.2.4.2.3(1)$$

Where ε_x as calculated above is less than zero ε_x shall either be taken as zero or recalculated as follows:

$$\varepsilon_x = \frac{\left| \frac{M^*}{d_v} \right| + \sqrt{\left(|V^*| - \gamma_p P_v \right)^2 + \left[\frac{0.9T^* u_h}{2A_o} \right]^2} + 0.5N^* - A_{pt}f_{po}}{2(E_s A_{st} + E_p A_{pt} + E_c A_{ct})} \quad \dots 8.2.4.2.3(2)$$

within the limits $-0.2 \times 10^{-3} \leq \varepsilon_x \leq 0$ and M^* and V^* are absolute values and where—

$$M^* \geq d_v \sqrt{\left(|V^*| - \gamma_p P_v \right)^2 + \left[\frac{0.9T^* u_h}{2A_o} \right]^2} \quad \dots 8.2.4.2.3(3)$$

N^* is taken as positive for tension and negative for compression.

For rigid frames and rectangular culverts, the value of N^* used to determine ε_x may be taken as twice the compressive axial thrust calculated by elastic analysis.

A_{st} and A_{pt} are the areas of reinforcing bars and prestressing tendons respectively in the half depth of the section containing the flexural tension zone (between $D/2$ and the extreme tensile fibre).

NOTE: f_{po} may be taken as $0.5f_{pb}$ or $\sigma_{p,ef}$ for bonded tendons outside of the transfer length and $\sigma_{p,ef}$ for unbonded tendons.

In calculating A_{st} , the area of bars that terminate less than their development length from the section under consideration shall be reduced in proportion to their lack of full development.

For sections closer than d_v to the face of the support, the value of ε_x calculated at d_v from the face of the support may be used in evaluating k_v and θ_v .

If the axial tension is large enough to crack the flexural compression face of the section, the resulting increase in ε_x shall be considered. In lieu of more accurate calculations, ε_x calculated from the equation shall be doubled.

k_v and θ_v may be determined from Clause 8.2.4.4 using a value of ε_x that is greater than that calculated from the equation in this Clause. The mid-depth strain parameter ε_x shall not be greater than 3.0×10^{-3} .

8.2.4.3 Determination of k_v and θ_v for non-prestressed components (simplified method)

For normal weight concrete with $f'_c \leq 65$ MPa, the angle of inclination of the compression strut (θ_v) may be taken as 36° , and the value of k_v may be determined as follows:

$$(a) \text{ For } \frac{A_{sv}}{s} < \frac{A_{sv,min}}{s} \text{ then } k_v = \frac{200}{(1000 + 1.3d_v)} \leq 0.15 \quad \dots 8.2.4.3(1)$$

$$(b) \text{ For } \frac{A_{sv}}{s} \geq \frac{A_{sv,min}}{s} \text{ then } k_v = 0.15 \quad \dots 8.2.4.3(2)$$

8.2.4.4 *Secondary effects on V_{uc}*

Where stresses due to secondary effects such as creep, shrinkage and differential temperature are more than 10% of the total, they shall be taken into account in the calculation of V_{uc} both for reinforced and prestressed concrete beams.

8.2.4.5 *Reversal of loads*

Where loading cases occur which result in cracking in a zone usually in compression, the value of V_{uc} obtained from Clause 8.2.4 may not apply. In this case, V_{uc} shall be assessed or be taken as zero.

8.2.5 **Transverse shear and torsion reinforcement contribution**

8.2.5.1 *General*

Where the spacing, (s), of the transverse shear reinforcement changes, the quantity (A_{sv}/s) may be assumed to vary linearly over a length, D , centred on the location where the spacing changes.

8.2.5.2 *Transverse reinforcement for shear*

The contribution to the design shear strength (V_u) by shear reinforcement in a beam (V_{us}) shall be determined from the following equations:

(a) For perpendicular shear reinforcement:

$$V_{us} = \left(\frac{A_{sv} f_{sy.f} d_v}{s} \right) \cot(\theta_v) \quad \dots 8.2.5.2(1)$$

(b) For inclined shear reinforcement:

$$V_{us} = \left(\frac{A_{sv} f_{sy.f} d_v}{s} \right) (\sin(\alpha_v) \cot(\theta_v) + \cos(\alpha_v)) \quad \dots 8.2.5.2(2)$$

where

α_v = angle between the inclined shear reinforcement and the longitudinal tensile reinforcement

θ_v = angle between the axis of the concrete compression strut and the longitudinal axis of the member calculated in accordance with Clause 8.2.4

8.2.5.3 *Transverse reinforcement for combined shear and torsion*

For sections subjected to combined shear and torsion, the transverse reinforcement that is provided shall be at least equal to the sum of that required for shear and that required for the coexisting torsion.

8.2.5.4 **[Text deleted]**

8.2.5.5 *Minimum torsional reinforcement*

Where torsional reinforcement is required as specified in Clause 8.2.3.1 or where torsional strength is not required for the equilibrium of the structure and Clause 8.2.1.2 applies—

- (a) longitudinal torsional reinforcement shall be provided in accordance with Clauses 8.2.7 and 8.2.8; and
- (b) minimum transverse reinforcement shall be provided to satisfy the greater of—
 - (i) the minimum shear reinforcement required by Clause 8.2.1.7 in the form of closed fitments; and
 - (ii) a torsional capacity equal to $0.25T_{cr}$, where T_{cr} is calculated using Equation 8.2.1.2(2).

8.2.5.6 Torsional resistance

The value of T_{us} shall be determined from the following equation:

$$T_{us} = 2A_o \frac{A_{sw} f_{sy,f}}{s} \cot \theta_v \quad \dots 8.2.5.6$$

where

A_o = area enclosed by the shear flow path, including any holes therein

For solid sections A_o shall be taken as equal to $0.85A_{oh}$.

A_{oh} = area enclosed by the centre-line of the exterior closed transverse torsion reinforcement

θ_v = as calculated in Clause 8.2.4

8.2.6 Hanging reinforcement

Loads applied to a member other than at the top chord of the member shall be transferred to the top chord, within the load application region, by the provision of hanging reinforcement of area consistent with strut-and-tie modelling.

8.2.7 Additional longitudinal tension forces caused by shear and torsion

The additional longitudinal force caused by shear and/or torsion shall be as follows:

$$\Delta F_{td} = \Delta F_{tds} + \Delta F_{tdt} \quad \dots 8.2.7(1)$$

where

(a) For shear:

$$\Delta F_{tds} = 0.5(V^* - \gamma_p P_v + \phi V_{uc}) \cot \theta_v, \text{ but not less than zero} \quad 8.2.7(2)$$

(b) For torsion:

$$\Delta F_{tdt} = 0.5T^* \frac{u_o}{2A_o} \cot \theta_v, \text{ but not less than zero} \quad 8.2.7(3)$$

and

γ_p is given in Clause 8.2.1.3 and ϕ is determined from Table 2.2.2(e).

u_o = area enclosed by the shear flow path, including any holes therein

For solid sections u_o shall be taken as equal to $0.92u_h$.

In Equation 8.2.7(2) the term $0.5(V^* - \gamma_p P_v + \phi V_{uc})$ shall not be taken as greater than $(V^* - \gamma_p P_v)$.

8.2.8 Proportioning longitudinal reinforcement

8.2.8.1 General

Reinforcement shall be provided to resist the sum of the tension forces including tension caused by flexure, shear, torsion and applied axial forces, and is shown in Figure 8.2.8.

All reinforcement shall be anchored to develop the determined forces. At end supports the force shall be developed past the face of the support.

NOTE: The longitudinal reinforcement due to torsion should be distributed evenly over the height of the section or evenly concentrated at the corners of the section.

8.2.8.2 Flexural tension side

- A2 | An area of reinforcement and tendons shall be provided to resist the total tension force at every section calculated from—

$$T_{td} = \left(\frac{M^*}{z} + \frac{N^*}{2} + \Delta F_{td} \right) \quad \dots 8.2.8.2(1)$$

- A2 | but need not be more than that required at the section with the maximum tension force demand for flexure, axial force and torsion.

In Equation 8.2.8.2(1) —

- (i) z is the internal lever arm between the centroids of the resultant flexural compression force acting on the section and the resultant flexural tension force; and
(ii) N^* is taken as positive for tension and negative for compression;

A2 | $A_{st}f_s + A_{pt}\sigma_{pu} \geq T_{td}/\phi \quad \dots 8.2.8.2(2)$

where f_s is the maximum tensile stress that can be developed in the reinforcement at the section under consideration, but not greater than the yield strength f_{sy} .

In Equation 8.2.8.2(2), ϕ is determined from Table 2.2.2(b), Table 2.2.2(c) and Table 2.2.2(d) for the section under consideration and where A_{st} is the tensile reinforcement in the tension half of the cross-section and A_{pt} is the sum of all of the prestressing steel at the cross-section.

NOTE: For reinforcement and tendons that are not fully anchored, see Clauses 13.1.2.3 and 13.3.

- A2 | Alternatively, for reinforced concrete members not designed for axial tension or torsion and where there are no sudden changes in the calculated tension force, the above may be deemed to be satisfied by extending the flexural tensile reinforcement by a distance $d_v \cot \theta_v$ (see Figure 8.2.8), where d_v and θ_v are taken at the section under consideration.

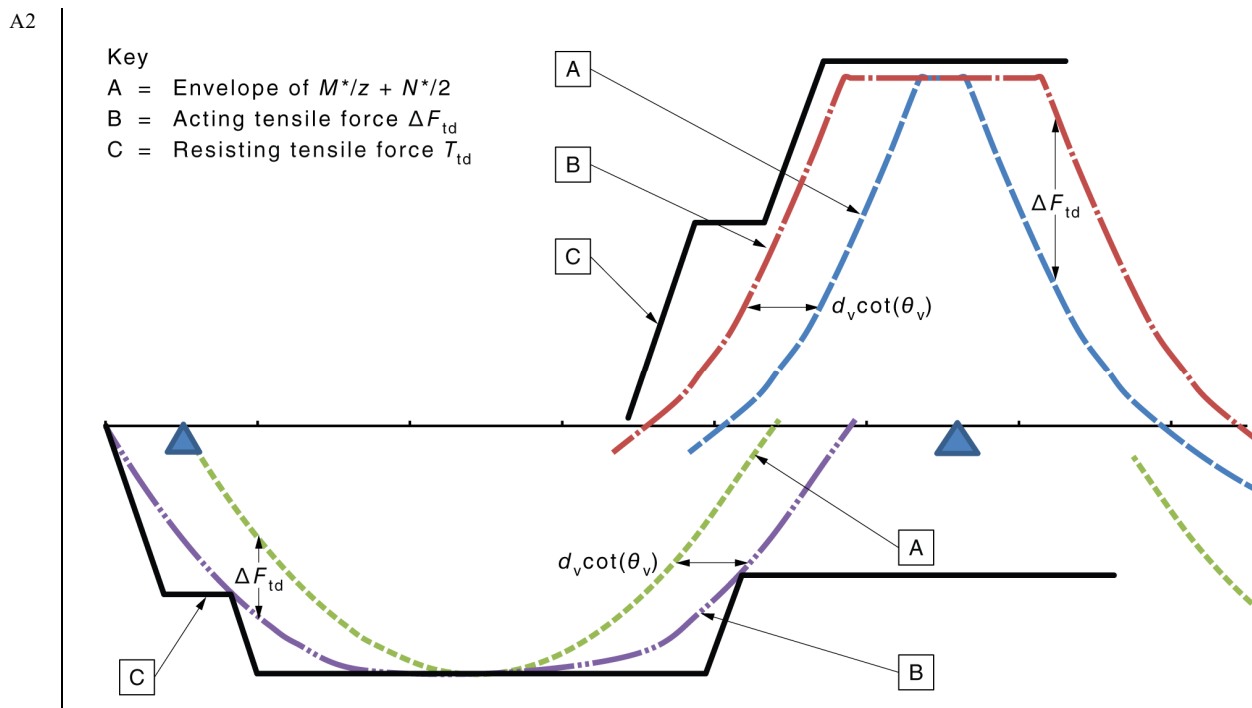


FIGURE 8.2.8 ILLUSTRATION OF THE CURTAILMENT OF LONGITUDINAL REINFORCEMENT AND TENDONS, TAKING INTO ACCOUNT THE EFFECT OF INCLINED CRACKS AND THE RESISTANCE OF REINFORCEMENT AND TENDONS WITHIN ANCHORAGE LENGTHS

8.2.8.3 Flexural compression side

At sections where there is a net tension force according to—

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$$T_{cd} = \left(-\frac{M^*}{z} + \frac{N^*}{2} + \Delta F_{ld} \right) \quad \dots 8.2.8.3(1)$$

reinforcing steel shall be provided such that—

$$(A_{sc} f_{sy}) \geq \frac{T_{cd}}{\phi} \quad \dots 8.2.8.3(2)$$

In Equation 8.2.8.3(2), ϕ is determined from Table 2.2.2(b)–(d) for the section under consideration and A_{sc} is the reinforcement in the compression half of the cross-section.

NOTES:

- 1 For minimum extent of reinforcement, see Section 8.3.
- 2 When detailing flexural reinforcement, designers should be cautious in D-regions and design accordingly.

8.3 GENERAL DETAILS FOR BEAMS

8.3.1 Detailing of flexural reinforcement and tendons

8.3.1.1 Distribution of reinforcement and integrity reinforcement

Tensile reinforcement shall be well distributed in zones of maximum concrete tension, including those portions of flanges of T-beams, L-beams and I-beams over a support.

For in-situ construction, the following shall constitute minimum requirements for integrity reinforcement passing through a support of a beam other than a perimeter beam:

- (a) At a continuous support, at least one Ductility Class N bottom bar of diameter not less than 20 mm, shall be continuous or shall be spliced with a tension lap splice or a mechanical or welded splice satisfying the requirements of Clause 13.2.
- (b) At non-continuous support, at least one Ductility Class N bottom bar of diameter not less than 20 mm, shall be anchored to develop f_{sy} at the face of the support using a standard hook satisfying Clause 13.1.2.7 or headed deformed bar satisfying Clause 13.1.4.

Beams along the perimeter of the structure shall have continuous reinforcement along the entire span and over continuous supports complying with (i) and (ii):

- (i) At least one-sixth of the tension reinforcement required for negative moment at the support, but not less than two bars.
- (ii) At least one-quarter of the tension reinforcement required for positive moment at the midspan, but not less than two bars.

The continuous reinforcement shall be enclosed by closed fitments anchored around a longitudinal bar in each corner. The fitments need not be extended through the continuous supports.

Where splices are required, the top reinforcement shall be spliced at or near midspan and bottom reinforcement shall be spliced at or near the support.

8.3.1.2 Continuation of negative moment reinforcement

Not less than one third of the total negative moment tensile reinforcement required at a support shall be extended a distance equal to the overall depth of the cross-section (D) beyond the point of contra-flexure.

8.3.1.3 Anchorage of positive moment reinforcement

Not less than one half of the tensile reinforcement required at midspan shall extend past the face of the support for a length of $12d_b$ plus a cog or an equivalent anchorage; or not less than one third of the tensile reinforcement required at midspan shall extend past the face of the support for a length of $8d_b$ plus $D/2$.

At a continuous or flexurally restrained support not less than one quarter of the total positive moment reinforcement required at midspan shall continue past the near face of the support.

8.3.1.4 Shear strength requirements near terminated flexural reinforcement

If tensile reinforcement is terminated, the effect on the shear strength shall be assessed in accordance with the principles of strut-and-tie modelling.

This requirement shall be deemed to be satisfied if any one of the following conditions is met:

- (a) Not more than a quarter of the maximum tensile reinforcement is terminated within any distance $2D$.
- (b) At the cut-off point, ' $\phi V_u \geq 1.5(V^* - \gamma_p P_v)$ '.
- (c) Fitments are provided to give an area of shear reinforcement of $A_{sv} + A_{sv.min}$ for a distance equal to the overall depth of the cross-section (D) along the terminated bar from the cut-off point, where A_{sv} and $A_{sv.min}$ are determined in accordance with Clause 8.2.

8.3.1.5 Deemed to comply arrangement of flexural reinforcement

For continuous reinforced beams designed using simplified methods of analysis, as detailed in Clause 6.10, the following arrangements are deemed to satisfy the requirements of Clauses 8.3.1.2 to 8.3.1.4:

- (a) Of the negative moment tensile reinforcement provided at the support—
 - (i) not less than one quarter shall extend over the whole span;
 - (ii) not less than one half shall extend $0.3L_n$ or more beyond the face of the support; and
 - (iii) the remainder, if any, shall extend $0.2L_n$ or more beyond the face of the support.

Where adjacent spans are unequal, the extension of negative reinforcement beyond each face of the common support shall be based on the longer span.
- (b) Of the positive moment tensile reinforcement provided at midspan—
 - (i) not less than one half shall extend into a simple support for a length of $12d_b$ with a cog;
 - (ii) not less than one quarter shall extend into a support where the beam is continuous or flexurally restrained; and
 - (iii) the remainder, if any, shall extend to within $0.1 L_n$ from the face of the support.
- (c) To comply with shear requirements, not more than a quarter of the maximum tensile reinforcement shall be terminated within any distance $2D$.

8.3.1.6 Restraint of compressive reinforcement

Compressive reinforcement required for strength in beams shall be adequately restrained by fitments in accordance with Clause 10.7.4.

8.3.1.7 Bundled bars

Groups of parallel longitudinal bars bundled to act as a unit shall—

- (a) have not more than four bars in any one bundle;
- (b) be tied together in contact; and
- (c) be enclosed within fitments.

Within the span of a flexural member, individual bars in a bundle shall be terminated so that the points of termination are staggered by a distance greater than or equal to 40 times the diameter of the larger bar within the bundle.

The unit of bundled bars shall be treated as an equivalent single bar of diameter derived from the total area of the bars in the bundle.

8.3.1.8 Detailing of tendons

In prestressed members—

- (a) anchorages and stress development, as appropriate, shall be provided for all tendons in accordance with Clause 12.5 and Section 13;
- (b) at a simple support of a pretensioned member, at least one third of the tendons required at the section of maximum positive moment shall be continued to the end of the member without debonding; and
- (c) for horizontal curvature of tendons, the designer shall assess the bursting and/or splitting capacity of the beam.

8.3.2 Detailing of shear and torsional reinforcement

8.3.2.1 General

Shear reinforcement shall comprise of—

- (a) fitments making an angle α_v of between of 45° and 90° with the longitudinal bars; or
- (b) welded wire mesh; or
- (c) in circular or oval-sectioned members, by helices.

NOTE: Straight reinforcement or tendons is permissible provided they are fully anchored at both the top and bottom of a member such that the design stress can be fully developed. Where stress in the shear reinforcement are greater than 500 MPa at the ultimate limit state, crack widths under service conditions should be considered.

8.3.2.2 Spacing

Shear reinforcement shall be spaced longitudinally not further apart than $0.5D$ or 300 mm, whichever is less. Where $V^* \leq \phi V_{u,\min}$, the spacing may be increased to $0.75D$ or 500 mm, whichever is less.

The maximum transverse spacing across the width of the member shall not exceed the lesser of 600 mm and D .

8.3.2.3 Extent

The shear reinforcement required at the critical cross-section shall be carried to the face of the support.

Shear reinforcement, of area not less than that calculated as being necessary at any cross-section, shall be provided for a distance (D) from that cross-section in the direction of decreasing shear. The first fitment at each end of a span shall be positioned not more than 50 mm from the face of the adjacent support.

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Shear reinforcement shall extend as close to the compression face and the tension face of the member as cover requirements and the proximity of other reinforcement and tendons will permit.

8.3.2.4 Anchorage of shear reinforcement

The anchorage of shear reinforcement transverse to the longitudinal flexural reinforcement shall be achieved by a hook or cog complying with Clause 13.1.2.7 or by welding of the fitment to a longitudinal bar or by a welded splice, or by lapped splices. Where lapped splices are used, the lap length shall be as calculated in Clause 13.1.2, except that for fitments adjacent to the cover concrete, the lap length calculated in Clause 13.1.2 shall be multiplied by 1.3.

Shear reinforcement shall be deemed to be anchored provided the following criteria are met:

- (a) Bends in bars used as fitments shall enclose a longitudinal bar with a diameter not less than the diameter of the fitment bar. The enclosed bar shall be in contact with the fitment bend.
- (b) The fitment spacing shall comply with Clause 8.3.2.2. Provided the hook comply with Clause 13.1.2.7, anchorage shall be deemed to be satisfied.
- (c) Fitment cogs shall not be used when the anchorage of the fitment is solely in the outer layer of reinforcement. In this case fitment hooks shall be used.

NOTE: The type of anchorage used should not induce splitting or spalling of the concrete cover. Notwithstanding the above, fitment cogs shall not to be used when the fitment cog is located within 50 mm of any concrete surface.

8.3.2.5 End anchorage of mesh

Where mesh is used as shear reinforcement, the ends shall be anchored—

- (a) in accordance with Clause 8.3.2.4, if the wires are bent at least to the dimensions of a standard fitment hook; or
- (b) by embedding two or more transverse wires at least 25 mm within the compressive zone.

8.3.3 Detailing of torsional reinforcement

Torsional reinforcement shall be detailed in accordance with Clause 8.3.2.4 and the following:

- (a) Torsional reinforcement shall consist of both closed fitments and longitudinal reinforcement and tendons.
- (b) The closed fitments shall be continuous around all sides of the cross-section and anchored so as to develop full strength at any point, unless a more refined analysis shows that full anchorage is not required over part of the fitment at each corner of the torsion cell to develop full strength. The spacing of the closed fitments shall be not greater than the lesser of $0.12u_h$ and 300 mm. In large members where a single closed loop of reinforcement is not possible, bars shall extend in one length over the full depth of the web, or width of the flange, with adequate anchorage by means of hooks or cogs at the intersection of webs and flanges.

Alternatively, fitments may be lapped. Lap lengths shall be as calculated in Clause 13.1.2, except that for fitments adjacent to the cover concrete the lap length calculated in Clause 13.1.2 shall be multiplied by 1.3.

- (c) Additional longitudinal reinforcement shall be placed as close as practicable to the corners of the cross section, and in all cases at least one longitudinal bar shall be provided at each corner of the closed ties.

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- (d) The section shall be capable of distributing axial torsional tensile force equally to the corners of the torsion cell.

8.4 LONGITUDINAL SHEAR IN COMPOSITE AND MONOLITHIC BEAMS

8.4.1 General

This clause applies to the transfer of longitudinal shear forces, across interface shear planes through webs and flanges of—

- (a) composite beams constructed of precast concrete sections and cast in situ toppings or flanges; and
- (b) beams constructed monolithically.

8.4.2 Design shear stress

The design shear stress (τ^*) acting on the interface shall be taken as follows:

$$\tau^* = \frac{\beta V^*}{z b_f} \quad \dots 8.4.2$$

where

z = internal moment lever arm

For a shear plane that passes through a region in compression—

β = ratio of the compressive force in the member (calculated between the extreme compressive fibre and the shear plane) and the total compression force in the section

For a shear plane that passes through a region in tension—

β = ratio of the tensile force in the longitudinal reinforcement (calculated between the extreme tensile fibre and the shear plane) and the total tension force in the section

8.4.3 Shear stress capacity

The design shear stress at the shear interface shall not exceed $\phi \tau_u$ where—

$$\tau_u = \mu \left(\frac{A_{sf} f_{sy}}{s b_f} + \frac{g_p}{b_f} \right) + k_{co} f'_{ct} \leq \text{lesser of } (0.2 f'_c, 10 \text{ MPa}) \quad \dots 8.4.3$$

where

τ_u = unit shear strength

g_p = permanent distributed load normal to the shear interface per unit length, newtons per millimetre (N/mm)

μ = coefficient of friction given in Table 8.4.3

k_{co} = cohesion coefficient given in Table 8.4.3

b_f = width of the shear plane, in millimetres (mm)

A_{sf} = area of fully anchored shear reinforcement crossing the interface (mm²)

f_{sy} = yield strength of shear reinforcement not exceeding 500 MPa

s = spacing of anchored shear reinforcement crossing interface

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TABLE 8.4.3
SHEAR PLANE SURFACE COEFFICIENTS

Surface condition of the shear plane	Coefficients	
	μ	k_{co}
A smooth surface, as obtained by casting against a form, or finished to a similar standard	0.6	0.1
A surface trowelled or tamped, so that the fines have been brought to the top, but where some small ridges, indentations or undulations have been left; slip-formed and vibro-beam screeded; or produced by some form of extrusion technique	0.6	0.2
A surface deliberately roughened— (a) by texturing the concrete to give a pronounced profile; (b) by compacting but leaving a rough surface with coarse aggregate protruding but firmly fixed in the matrix; (c) by spraying when wet, to expose the coarse aggregate without disturbing it	0.7	0.4
Monolithic construction or mechanical shear keys.	0.9	0.5

NOTE: Where a beam is subjected to high levels of differential shrinkage, temperature effects, tensile stress or fatigue effects across the shear plane, the values of μ and k_{co} in the above table do not apply.

8.4.4 Shear plane reinforcement

Where reinforcement is required to increase the longitudinal shear strength, the reinforcement shall consist of shear reinforcement anchored to develop its full strength at the shear plane. Shear and torsional reinforcement already provided, and which crosses the shear plane, may be taken into account for this purpose.

The centre-to-centre spacing (s) of the shear reinforcement shall not exceed the maximum spacing—

$$s_{\max} = 3.5t_f \quad \dots 8.4.4$$

where

t_f = thickness of topping or flange anchored by the shear reinforcement

8.4.5 Minimum thickness of structural components

The average thickness of structural components subject to interface shear shall be not less than 50 mm with a minimum local thickness not less than 30 mm.

8.5 DEFLECTION OF BEAMS

8.5.1 General

The deflection of a beam shall be determined in accordance with Clause 8.5.2 or Clause 8.5.3.

Alternatively, for reinforced beams, the effective-span to effective-depth ratio shall conform with Clause 8.5.4.

8.5.2 Beam deflection by refined calculation

The calculation of the deflection of a beam by refined calculation shall make allowance for the following:

- (a) Cracking and tension stiffening.
- (b) Shrinkage and creep properties of the concrete.
- (c) Expected load history.

- (d) Expected construction procedure.
- (e) Deflection of formwork or settlement of props during construction, particularly when the beam formwork is supported on suspended floors or beams below.

8.5.3 Beam deflection by simplified calculation

8.5.3.1 Short-term deflection

The short-term deflections due to external loads and prestressing, which occur immediately on their application, shall be calculated using the value of E_{cj} determined in accordance with Clause 3.1.2 and the value of the effective second moment of area of the member (I_{ef}). This value of I_{ef} shall be determined by rational calculation. Alternatively, I_{ef} may be determined at the nominated cross-sections as follows:

- (a) For a simply supported span, the value at midspan.
- (b) In a continuous beam—
 - (i) for an interior span, half the midspan value plus one quarter of each support value; or
 - (ii) for an end span, half the midspan value plus half the value at the continuous support.
- (c) For a cantilever, the value at the support.

For the purpose of the above determinations, the value of I_{ef} at each of the cross-sections nominated in Items (a) to (c) above is given by—

$$I_{ef} = \frac{I_{cr}}{1 - \left(1 - \frac{I_{cr}}{I}\right) \left(\frac{M_{cr,t}}{M_s^*}\right)^2} \leq I_{ef,max} \quad \dots 8.5.3.1(1)$$

where

$I_{ef,max}$ = maximum effective second moment of area and is taken as I , for reinforced sections when $p = A_{st}/bd \geq 0.005$ for prestressed sections

= $0.6 I$, for reinforced sections when $p = A_{st}/bd < 0.005$

b = width of the cross-section at the compression face

M_s^* = maximum bending moment at the section, based on the short-term serviceability load or the construction load

$$M_{cr,t} = Z(f'_{ct,f} - \sigma_{cs} + P/A_g) + Pe \geq 0$$

Z = section modulus of the uncracked section, referred to the extreme fibre at which cracking occurs

$f'_{ct,f}$ = characteristic flexural tensile strength of concrete

σ_{cs} = maximum shrinkage-induced tensile stress on the uncracked section at the extreme fibre at which cracking occurs. In the absence of more refined calculation, the value of σ_{cs} that accounts for the restraint provided by the steel reinforcement may be taken as—

$$= \frac{2.5p_w - 0.8p_{cw}}{1 + 50p_w} E_s \varepsilon_{cs}$$

p_w = web reinforcement ratio for tensile reinforcement
 $= (A_{st} + A_{pt})/b_w d$

p_{cw} = web reinforcement ratio for compressive reinforcement = $A_{sc}/b_w d$

ε_{cs} = final design shrinkage strain of concrete at the time when deflection is to be calculated

If the short-term portion of the final long-term deflection is to be calculated ε_{cs} is the final long-term shrinkage strain determined in accordance with Clause 3.1.7. In the case of indeterminate members, σ_{cs} shall also account of the tension caused by the redundant actions induced by restraint.

Alternatively, for reinforced members, I_{ef} may be taken as—

$$I_{ef} = \left[(5 - 0.04 f'_c) p + 0.002 \right] b_{ef} d^3 \leq \left[0.1 / \beta^{2/3} \right] b_{ef} d^3 \text{ when } p \geq 0.001 (f'_c)^{1/3} / \beta^{2/3} \quad \dots 8.5.3.1(2)$$

$$I_{ef} = \left[\left(0.055 (f'_c)^{1/3} \right) / \beta^{2/3} - 50 p \right] b_{ef} d^3 \leq \left[0.06 / \beta^{2/3} \right] b_{ef} d^3 \text{ when } p < 0.001 (f'_c)^{1/3} / \beta^{2/3} \quad \dots 8.5.3.1(3)$$

where

$$\beta = b_{ef} / b_w \geq 1$$

$$p = A_{st} / (b_{ef} d) \text{ at midspan}$$

8.5.3.2 Long-term deflection

For reinforced and prestressed beams, that part of the deflection that occurs after the short term deflection shall be calculated as the sum of—

- the shrinkage component of the long-term deflection, determined from the design shrinkage strain of concrete (ε_{cs}) (see Clause 3.1.7) and the principles of mechanics; and
- the additional long-term creep deflections, determined from the design creep coefficient of concrete (φ_{cc}) (see Clause 3.1.8) and the principles of mechanics.

In the absence of more accurate calculations, the additional long-term deflection of a reinforced beam due to creep and shrinkage may be estimated by multiplying the short-term deflection caused by the sustained loads (obtained using the final long-term shrinkage strain in the estimate of $M_{cr,t}$) by a multiplier, k_{cs} , given by—

$$k_{cs} = \left[2 - 1.2 (A_{sc} / A_{st}) \right] \geq 0.8 \quad \dots 8.5.3.2$$

where A_{sc} is the area of steel in the compressive zone of the cracked section between the neutral axis at service loads and the extreme concrete compressive fibre and A_{sc}/A_{st} is taken at midspan, for a simply supported or continuous beam and at the support, for a cantilever beam.

8.5.4 Deemed to conform span-to-depth ratios for reinforced beams

For reinforced beams of uniform cross-section, fully propped during construction, subject to uniformly distributed loads only and where the imposed action (live load) (q) does not exceed the permanent action (dead load) (g), beam deflections shall be deemed to conform with the requirements of Clause 2.3.2 if the ratio of effective span to effective depth satisfies the following:

$$L_{ef} / d \leq \left[\frac{k_1 (\Delta / L_{ef}) b_{ef} E_c}{k_2 F_{d,ef}} \right]^{1/3} \quad \dots 8.5.4$$

where

Δ/L_{ef} = deflection limit selected in accordance with Clause 2.3.2(a)

L_{ef} = effective span

$F_{d,ef}$ = effective design load per unit length, taken as—

- (a) $(1.0 + k_{cs})g + (\psi_s + k_{cs} \psi_l)q$ for total deflection; or
- (b) $k_{cs} g + (\psi_s + k_{cs} \psi_l)q$ for the deflection that occurs after the addition or attachment of the brittle partitions or finishes

where k_{cs} is determined in accordance with Clause 8.5.3.2 and ψ_s and ψ_l are given in AS/NZS 1170.0

k_1 = $I_{ef}/b_{ef}d^3$, taken as—

$$(5 - 0.04f'_c)p + 0.002 \leq 0.1 / \beta^{2/3} \quad \text{when } p \geq 0.001(f'_c)^{1/3} / \beta^{2/3}$$

$$0.055(f'_c)^{1/3} / \beta^{2/3} - 50p \leq 0.06 / \beta^{2/3} \quad \text{when } p < 0.001(f'_c)^{1/3} / \beta^{2/3}$$

where $\beta = b_{ef}/b_w \geq 1$ and $p = A_{st}/(b_{ef} d)$ at midspan

k_2 = deflection constant, taken as—

- (a) for simply supported beams, 5/384; or
- (b) for continuous beams, where the ratio of the longer to the shorter of two adjacent spans does not exceed 1.2 and where no end span is longer than an interior span—
 - (i) 2.4/384 in an end span; or
 - (ii) 1.5/384 in interior spans.

NOTE: E_c is in megapascals.

8.6 CRACK CONTROL OF BEAMS

8.6.1 General requirements

Cracking is normal in reinforced concrete structures subjected to bending, shear, torsion or axial tension caused by either direct loading or restraint or imposed deformation. Cracking shall be limited to an extent that does not impair the durability or serviceability of the structure, both in terms of function and appearance. A characteristic maximum crack width w'_{max} shall be selected for each surface of a structure taking into account the function and exposure of the surface. The characteristic maximum crack width is the maximum limit imposed on the calculated crack width.

For regions of beams fully enclosed within a building except for a brief period of weather exposure during construction, and where it is assessed that cracking will not impair the functioning of the structure, only Items (a) and (b) need be satisfied:

- (a) The minimum area of reinforcement in a tensile zone of a beam shall conform with Clause 8.1.6.1.
- (b) The distance from the side or soffit of a beam to the centre of the nearest longitudinal bar shall not exceed 100 mm. Bars with a diameter less than half the diameter of the largest bar in the section shall be ignored. The centre-to-centre spacing of bars near a tension face of the beam shall not exceed 300 mm. For T-beams and L-beams, the reinforcement required in the flange shall be distributed across the effective width.

In other situations, in addition to (a) and (b) above, crack widths may be calculated and limited to the selected value of w'_{max} in accordance with Clause 8.6.2.3. A simplified alternative approach is to limit the size and spacing of reinforcement crossing the cracks in accordance with—

A2

A2

- A2 | (i) Clause 8.6.2 for reinforced concrete sections and sections containing unbonded tendons;
- (ii) Clause 8.6.3 for sections containing bonded tendons; and
- (iii) Clause 16.4.7.4 for sections containing steel fibres.

Under direct loading, the calculated tensile steel stress ($\sigma_{scr.1}$) shall not exceed $0.8f_{sy}$. Bars with a diameter less than half the diameter of the largest bar in the section shall be ignored when determining spacing.

NOTES:

- 1 It is possible that the width of some cracks in the actual structure may exceed the maximum characteristic value specified in design.
- 2 Design bending moments at the serviceability limit state will normally be estimated using elastic analysis. Substantial errors may result where the actual in-service moments are likely to have redistributed significantly from the elastic distribution.

8.6.2 Crack control for tension and flexure in reinforced beams

8.6.2.1 General

For the purpose of this clause the resultant action is considered to be *primarily tension* when the whole of the section is in tension, or *primarily flexure* when the tensile stress distribution within the section prior to cracking is triangular with some part of the section in compression.

8.6.2.2 Crack control without direct calculation of crack widths

A2 | The calculated steel stress (σ_{scr}) on the cracked section shall comply with the following:

- (a) For beams primarily subject to tension, σ_{scr} shall not exceed the maximum steel stress given in Table 8.6.2.2(A) for the largest nominal diameter (d_b) of the bars in the section.
- (b) For beams primarily subject to flexure, σ_{scr} shall not exceed the larger of the maximum steel stresses given in—
 - (i) Table 8.6.2.2(A) for the largest nominal diameter (d_b) of the bars in the tensile zone; and
 - (ii) Table 8.6.2.2(B) for the largest centre-to-centre spacing of adjacent parallel bars in the tensile zone.

Under direct loading the calculated tensile steel stress ($\sigma_{scr.1}$) shall not exceed $0.8f_{sy}$. Bars with a diameter less than half the diameter of the largest bar in the section shall be ignored when determining spacing.

A2 | NOTE: [Text deleted]

TABLE 8.6.2.2(A)
MAXIMUM STEEL STRESS FOR TENSION
OR FLEXURE IN REINFORCED BEAMS

Nominal bar diameter (d_b) mm	Maximum steel stress MPa		
	$w'_{\max} = 0.2 \text{ mm}$	$w'_{\max} = 0.3 \text{ mm}$	$w'_{\max} = 0.4 \text{ mm}$
10	190	265	335
12	175	245	305
16	155	215	270
20	140	195	240
24	125	175	215
28	115	160	200
32	105	150	185
36	100	140	175
40	90	130	165

TABLE 8.6.2.2(B)
MAXIMUM STEEL STRESS FOR FLEXURE
IN REINFORCED BEAMS

Centre-to-centre spacing mm	Maximum steel stress MPa		
	$w'_{\max} = 0.2 \text{ mm}$	$w'_{\max} = 0.3 \text{ mm}$	$w'_{\max} = 0.4 \text{ mm}$
50	200	300	400
100	170	270	360
150	155	245	330
200	145	225	300
250	135	210	280
300	125	200	260

8.6.2.3 Crack control by calculation of crack widths

The calculated maximum crack width in a reinforced concrete member at any time after cracking is given by:

$$w = s_{r,\max} (\varepsilon_{sm} - \varepsilon_{cm}) \leq w'_{\max} \quad \dots 8.6.2.3(1)$$

where

$s_{r,\max}$ = the maximum crack spacing;

ε_{sm} = the mean strain in the reinforcement at the design loads for the serviceability limit states, including the effects of tension stiffening and any imposed deformations

ε_{cm} = the mean strain in the concrete between the cracks.

The difference between the mean strain in the reinforcement and the mean strain in the concrete is taken as:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{scr}}{E_s} - \frac{0.6 f_{ct}}{E_s p_{eff}} (1 + n_e p_{eff}) + \varepsilon_{cs} \geq 0.6 \frac{\sigma_{scr}}{E_s} \quad \dots 8.6.2.3(2)$$

where

σ_{scr} = the stress in the tensile reinforcement assuming a cracked section

ε_{cs} = the absolute value of the final long-term shrinkage strain determined in

accordance with Clause 3.1.7

n_e = the effective modular ratio $(1 + \varphi_{cc})E_s/E_c$

φ_{cc} = the creep coefficient associated with the time interval after cracking determined in accordance with Clause 3.1.8

f_{ct} = the mean value of the axial tensile strength of concrete at the time cracking is expected

p_{eff} = the reinforcement ratio given by $A_{st}/A_{c,eff}$

$A_{c,eff}$ = the effective area of concrete in tension surrounding the bars with depth $h_{c,eff}$ equal to the lesser of $2.5(D-d)$, $(D-kd)/3$ or $D/2$

D = the overall depth of the cross-section

d = the effective depth to the centroid of the tensile steel

kd = the depth of the neutral axis on the cracked section

For cross-sections with bonded reinforcement fixed at reasonably close centres, i.e. bar spacing $\leq 5(c + 0.5d_b)$, the maximum final crack spacing may be calculated from:

$$s_{r,max} = 3.4c + 0.3 k_1 k_2 d_b / p_{eff} \leq 1.3(D - kd) \quad \dots 8.6.2.3(3)$$

where

d_b = the tensile bar diameter

Where a mixture of bar diameters is used in a section, an equivalent bar diameter ($d_{b,eq}$) shall be used. For a section with n_1 bars of diameter d_{b1} and n_2 bars of diameter d_{b2} , the following expression shall be used:

$$d_{b,eq} = \frac{n_1 d_{b1}^2 + n_2 d_{b2}^2}{n_1 d_{b1} + n_2 d_{b2}} \quad \dots 8.6.2.3(4)$$

c = the clear cover to the longitudinal reinforcement

k_1 = a coefficient that accounts for the bond properties of the bonded reinforcement, with $k_1 = 0.8$ for deformed and $k_1 = 1.6$ for plain bars

k_2 = a coefficient that accounts for the longitudinal strain distribution, with $k_2 = 0.5$ for bending and $k_2 = 1.0$ for pure tension. For cases in combined tension and bending, $k_2 = (\varepsilon_1 + \varepsilon_2)/(2\varepsilon_1)$ and ε_1 is the greater and ε_2 is the lesser of the tensile strains at the boundaries of the cross-section (assessed on the basis of a cracked section)

8.6.3 Crack control for flexure in prestressed beams

Flexural cracking in a prestressed beam shall be deemed to be controlled if, under the short-term service loads, the resulting maximum tensile stress in the concrete does not exceed $0.25\sqrt{f'_c}$ or, if this stress is exceeded, by providing reinforcement or bonded tendons, or both, near the tensile face with a centre-to-centre spacing not exceeding 300 mm and by either of the following:

- (a) Limiting the calculated maximum flexural tensile stress in the concrete under short-term service loads to $0.6\sqrt{f'_c}$.

or

- (b) Limiting the increment in steel stress near the tension face to that given in Table 8.6.3, as the load increases from its value when the extreme concrete tensile fibre is at zero stress to the short-term service load value.

or

- (c) Limiting the calculated maximum crack width in accordance with Clause 8.6.2.3.

TABLE 8.6.3

Nominal bar diameter (d_b) mm	Maximum increment of steel stress for flexure in prestressed beams MPa		
	$w'_{\max} = 0.2 \text{ mm}$	$w'_{\max} = 0.3 \text{ mm}$	$w'_{\max} = 0.4 \text{ mm}$
≤ 12	240	300	350
16	200	260	300
20	170	220	260
24	145	195	225
≥ 28	120	185	210
All bonded tendons	120	185	210

8.6.4 Crack control in the side face of beams

For crack control in the side face of beams where the overall depth exceeds 750 mm, longitudinal reinforcement, consisting of 12 mm bars at 200 mm centres or 16 mm bars at 300 mm centres, shall be placed in each side face.

8.6.5 Crack control at openings and discontinuities

Reinforcement shall be provided for crack control at openings and discontinuities in a beam.

8.7 VIBRATION OF BEAMS

Vibration of beams shall be considered and action taken, where necessary, to ensure that vibrations induced by machinery, or vehicular or pedestrian traffic, will not adversely affect the serviceability of the structure.

8.8 T-BEAMS AND L-BEAMS

8.8.1 General

Where a slab is assumed to provide the flange of a T-beam or L-beam, the longitudinal shear capacity of the flange-web connection shall be checked in accordance with Clause 8.4.

For isolated T-beams or L-beams, the shear strength of the slab flange on vertical sections parallel to the beam shall also be checked in accordance with Clause 8.2.

8.8.2 Effective width of flange for strength and serviceability

In the absence of a more accurate determination, the effective width of the flange of a T beam or L-beam for strength and serviceability shall be taken as—

- (a) T-beams..... $b_{\text{ef}} = b_w + 0.2a$; and
 (b) L-beams..... $b_{\text{ef}} = b_w + 0.1a$,

where a is the distance between points of zero bending moment, which, for continuous beams, may be taken as $0.7L$.

In both Items (a) and (b) above, the overhanging part of the flange considered effective shall not exceed half the clear distance to the next member. The effective width so determined may be taken as constant over the entire span.

8.9 SLENDERNESS LIMITS FOR BEAMS

8.9.1 General

Unless a stability analysis is carried out, beams shall conform with the limits specified in Clauses 8.9.2 to 8.9.4.

8.9.2 Simply supported and continuous beams

For a simply supported or continuous beam, the distance L_1 between points at which lateral restraint is provided shall be such that L_1/b_{ef} does not exceed the lesser of $180 b_{ef}/D$ and 60.

8.9.3 Cantilever beams

For a cantilever beam having lateral restraint only at the support, the ratio of the clear projection (L_n) to the width (b_{ef}) at the support shall be such that L_n/b_{ef} does not exceed the lesser of $100 b_{ef}/D$ and 25.

8.9.4 Reinforcement for slender prestressed beams

For a prestressed beam in which L_1/b_{ef} exceeds 30, or for a prestressed cantilever beam in which L_n/b_{ef} exceeds 12, the following reinforcement shall be provided:

- (a) Stirrups providing a steel area, $A_{sv.min}$ in accordance with Clause 8.2.1.7.
- (b) Additional longitudinal reinforcement, consisting of at least one bar in each corner of the compression face, such that—

$$A_{sc} \geq 0.35 A_{pt} f_{pb}/f_{sy} \quad \dots 8.9.4$$

SECTION 9 DESIGN OF SLABS FOR STRENGTH AND SERVICEABILITY

9.1 STRENGTH OF SLABS IN BENDING

9.1.1 General

The strength of a slab in bending shall be determined in accordance with Clauses 8.1.1 to 8.1.8, except that for two-way reinforced slabs, the minimum strength requirements of Clause 8.1.6.1 shall be deemed to be satisfied by providing tensile reinforcement such that A_{st}/bd is not less than the following in each direction:

- (a) Slabs supported by columns at their corners $0.24(D/d)^2 f'_{ct,f} / f_{sy}$.
- (b) Slabs supported by beams or walls on four sides $0.19(D/d)^2 f'_{ct,f} / f_{sy}$.

9.1.2 Reinforcement and tendon distribution in two-way flat slabs

In two-way flat slabs, at least 25% of the total of the design negative moment in a column-strip and adjacent half middle-strips shall be resisted by reinforcement or tendons or both, located in a cross-section of slab centred on the column and of a width equal to twice the overall depth of the slab or drop panel plus the width of the column.

9.1.3 Detailing of tensile reinforcement in slabs

9.1.3.1 General procedure for arrangement

Tensile reinforcement shall be arranged in accordance with the following:

- (a) Where the bending moment envelope has been calculated, the termination and anchorage of flexural reinforcement shall be based on a hypothetical bending-moment diagram formed by displacing the calculated positive and negative bending-moment envelopes a distance D along the slab from each side of the relevant sections of maximum moment. Additionally, the following shall apply:
 - (i) Not less than one third of the total negative moment reinforcement required at a support shall be extended a distance $12d_b$ or D , whichever is greater, beyond the point of contraflexure.
 - (ii) At a simply supported discontinuous end of a slab, not less than one half of the total positive moment reinforcement required at midspan shall be anchored by extension past the face of the support for a distance of $12d_b$ or D , whichever is greater, or by an equivalent anchorage.

Where no shear reinforcement is required in accordance with Clause 8.2.5 or Clause 9.2, the extension of the midspan positive moment reinforcement past the face of the support may be reduced to $8d_b$ if at least one half of the reinforcement is so extended, or to $4d_b$ if all the reinforcement is so extended.
 - (iii) At a support where the slab is continuous or flexurally restrained, not less than one quarter of the total positive moment reinforcement required at midspan shall continue past the near face of the support.
 - (iv) Where frames incorporating slabs are designed to resist lateral loading, the effects of such loading on the arrangement of the slab reinforcement shall be taken into account but in no case shall the lengths of reinforcement be made less than those shown in Figures 9.1.3.2 and 9.1.3.4.

- (b) Where the bending moment envelope has not been calculated, the requirements of Clauses 9.1.3.2, 9.1.3.3 or 9.1.3.4, as appropriate to the type of slab, shall be satisfied.

9.1.3.2 Deemed-to-conform arrangement for one-way slabs

For one-way slabs continuous over two or more spans analysed using simplified elastic analysis, as detailed in Clause 6.10.2, where—

- (a) the ratio of the longer to the shorter of any two adjacent spans does not exceed 1.2; and
- (b) the imposed actions (live loads) may be assumed to be uniformly distributed and the imposed action (live load) (q) is not greater than twice the permanent action (dead load) (g),

the arrangement of tensile reinforcement shown in Figure 9.1.3.2 shall be deemed to conform with Clause 9.1.3.1(a).

Where adjacent spans are unequal, the extension of negative moment reinforcement beyond each face of the common support shall be based on the longer span.

For one-way slabs of single span, the arrangement of tensile reinforcement shown in Figure 9.1.3.2, for the appropriate end support conditions, shall be deemed to conform with Clause 9.1.3.1(a).

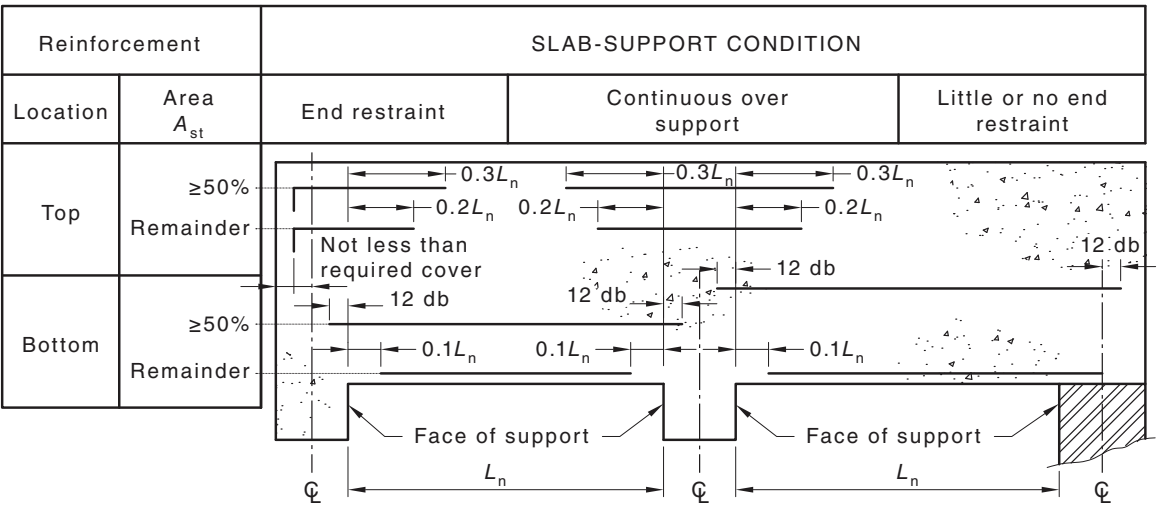


FIGURE 9.1.3.2 ARRANGEMENT OF REINFORCEMENT

9.1.3.3 Deemed-to-conform arrangement for two-way slabs supported on beams or walls

For two-way simply supported or continuous rectangular slabs supported by walls or beams on four sides analysed using simplified elastic analysis, as detailed in Clause 6.10.3, the following deemed-to-conform arrangement shall be used and the arrangement of tensile reinforcement, shown in Figure 9.1.3.2 and further prescribed herein, shall be deemed to conform with Clause 9.1.3.1(a):

- (a) The arrangement shall apply to each direction.
- (b) Where a simply supported or continuous slab is not square, the arrangement shall be based on the span (L_n) taken as the shorter span.

- (c) Where adjacent continuous rectangular slabs have unequal shorter spans, the extension of negative moment reinforcement beyond each face of a common support shall be based on the span (L_n) taken as the longer of the shorter spans.
- (d) Negative moment reinforcement provided at a discontinuous edge shall extend from the face of the support into the span for a distance of 0.15 times the shorter span.
- (e) At an exterior corner of a two-way rectangular slab supported on four sides and restrained against uplift, reinforcement shall be provided in both the top and the bottom of the slab. This reinforcement shall consist of two layers perpendicular to the edges of the slab and extend from each edge for a distance not less than 0.2 times the shorter span. The area of the reinforcement in each of the four layers shall be not less than—
 - (i) for corners where neither edge is continuous $0.75A_{st}$; and
 - (ii) for corners where one edge is continuous $0.5A_{st}$,
 where A_{st} is the area of the maximum positive moment reinforcement required at midspan.

Any reinforcement provided may be considered as part of this reinforcement.

9.1.3.4 Deemed-to-conform arrangement for two-way flat slabs

For multispan, reinforced, two-way flat slabs analysed using simplified elastic analysis, as detailed in Clause 6.10.4, the following deemed to conform arrangement shall be used and the arrangement of tensile reinforcement, shown in Figure 9.1.3.4 and further prescribed herein, shall be deemed-to-conform with Clause 9.1.3.1(a).

Where adjacent spans are unequal, the extension of negative moment reinforcement beyond each face of the common support shall be based on the longer span.

All slab reinforcement perpendicular to a discontinuous edge shall be extended (straight, bent or otherwise) past the internal face of the spandrel, wall or column for a length—

- (a) *for positive moment reinforcement*, not less than 150 mm except that it shall extend as close as permitted to the edge of the slab if there is no spandrel beam or wall; and
- (b) *for negative moment reinforcement*, such that the calculated force is developed at the internal face in accordance with Clause 13.1.

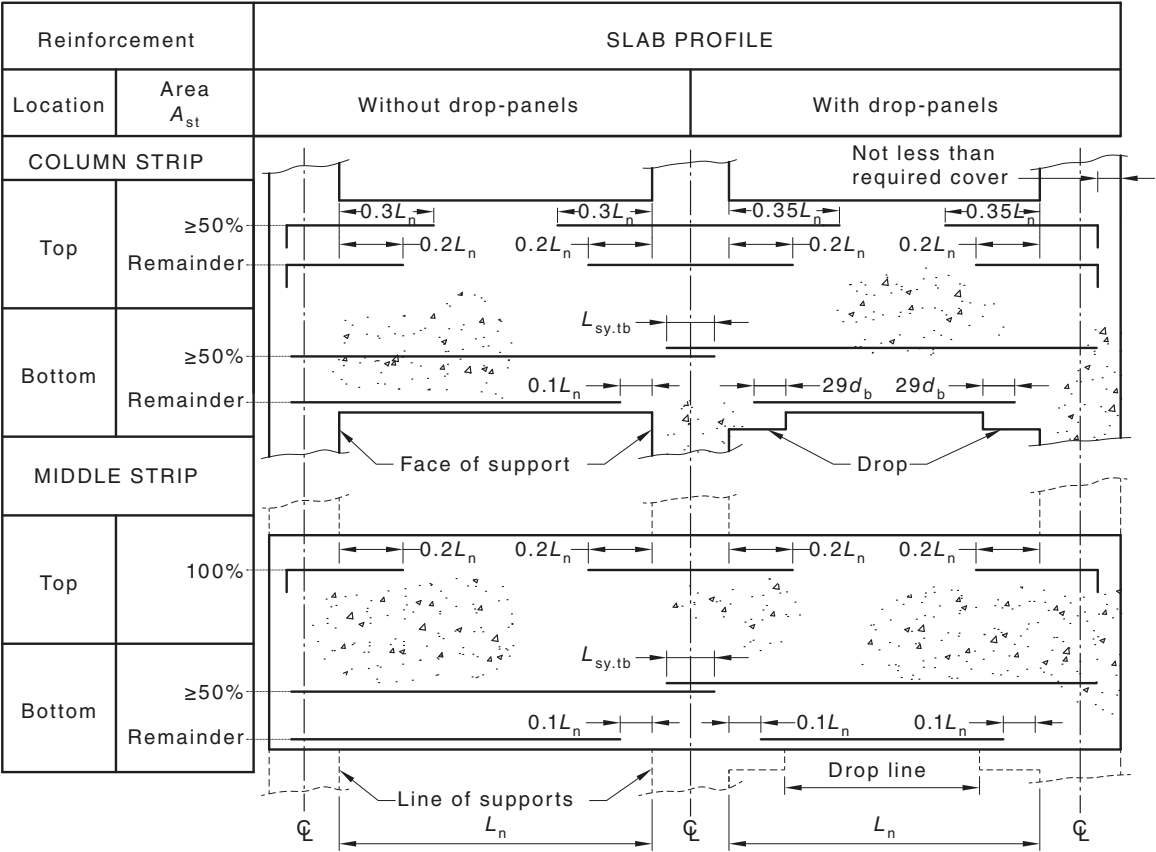


FIGURE 9.1.3.4 ARRANGEMENT OF REINFORCEMENT

9.2 STRUCTURAL INTEGRITY REINFORCEMENT

9.2.1 General

Reinforcement at connections shall be provided to increase the resistance of the structural system to progressive collapse at walls and columns.

At least two of the column strip bottom bars or strands in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be continuous through the interior support and fully anchored beyond the face of the exterior support.

9.2.2 Minimum structural integrity reinforcement

The summation of the area of bottom reinforcement connecting the slab, drop panel, or slab band to the column or column capital on all faces of the periphery of a column or column capital shall be not less than,

$$A_{s,min} = \frac{2N^*}{\phi f_{sy}} \quad \dots 9.2.2$$

in which N^* is the column reaction from the floor slab at the ultimate limit state.

Integrity reinforcement shall not be required if there are beams containing shear reinforcement and with at least two bottom bars continuous through the joint in all spans framing into the column.

This reinforcement shall be placed within the column core and may consist of the following:

- (a) Bottom reinforcement extended to lap over the column in accordance with Clause 13.2.

- (b) Extra bars over the column extending a minimum distance of $2 L_{sy.tb}$ from the face of the column or column capital and lapping with any existing bottom reinforcement. These bars should have hooked or cogged ends where terminating at discontinuous edges.

9.2.3 Minimum reinforcement for distributing loads

Minimum reinforcement in a secondary direction shall be provided for the purpose of distributing loads.

NOTE: For shrinkage and temperature effects see Clause 9.5.3.

9.2.4 Spacing of reinforcement and tendons

The minimum clear distance between parallel bars (including bundled bars), ducts and tendons shall be such that the concrete can be properly placed and compacted in accordance with Clause 17.1.3.

The maximum spacing of reinforcement and tendons for crack control shall be determined in accordance with Clause 9.5.

Where the plain concrete between tendons is unable to safely distribute the applied loads to the tendons, reinforcement for this purpose shall be supplied transverse in a prestressed slab. Unless calculations show that a wider spacing is possible, the maximum spacing of tendons in an unreinforced slab subjected to uniformly distributed loads shall be the lesser of 10 times the slab thickness and 1500 mm.

NOTE: The tendon spacing adjacent to a supporting column will be controlled by Clause 9.1.2 and may need to be significantly less than the above limits.

9.3 STRENGTH OF SLABS IN SHEAR

9.3.1 Definitions and symbols

For the purpose of this clause, the definitions and symbols below apply to flat slabs.

9.3.1.1 *Effective area of a support or concentrated load*

The area totally enclosing the actual support or load and for which the perimeter is a minimum [see Figure 9.3(A)].

9.3.1.2 *Critical opening*

Any opening through the thickness of a slab where an edge, or part of the edge, of the opening is located at a clear distance of less than $2.5b_o$ from the critical shear perimeter [see Figure 9.3(A)].

9.3.1.3 *Critical shear perimeter*

The perimeter defined by a line geometrically similar to the boundary of the effective area of a support or concentrated load and located at a distance of $d_{om}/2$ therefrom [see Figure 9.3(A)].

9.3.1.4 *Symbols*

The following symbols apply:

- a = dimension of the critical shear perimeter measured parallel to the direction of M_v^* [see Figure 9.3(B)]
- b_o = dimension of an opening [see Figure 9.3(A)]
- b_w = width of the web of a spandrel beam [see Figure 9.3(B)]
- D_b = overall depth of a spandrel beam [see Figure 9.3.6]
- D_s = overall depth of a slab or drop panel

- d_{om} = mean value of d_o , averaged around the critical shear perimeter
 M_v^* = bending moment transferred from the slab to a support in the direction being considered [see Figure 9.3(B)]
 u = length of the critical shear perimeter [see Figure 9.3(A)]
 y_1 = larger overall dimension of a closed fitment (see Figure 9.3.6)
 β_h = ratio of the longest overall dimension of the effective loaded area, Y , to the overall dimension, X , measured perpendicular to Y [see Figure 9.3(A)]

9.3.1.5 Torsion strip

A strip of slab of width a , whose longitudinal axis is perpendicular to the direction of M_v^* [see Figure 9.3(B)].

9.3.2 Strength

The strength of a slab in shear shall be determined in accordance with the following:

- (a) Where shear failure can occur across the width of the slab, the design shear strength of the slab shall be calculated in accordance with Clause 8.2.
- (b) Where shear failure can occur locally around a support or concentrated load, the design shear strength of the slab shall be taken as ϕV_u , where V_u is calculated in accordance with one of the following:
 - (i) Where M_v^* is zero, V_u is taken as equal to V_{uo} calculated in accordance with Clause 9.3.3.
 - (ii) Where M_v^* is not zero, V_u is calculated in accordance with Clause 9.3.4.

NOTE: For types of shear reinforcement other than those covered in Clauses 9.3.3 and 9.3.4 strength may be determined by tests, in accordance with Appendix B.

9.3.3 Ultimate shear strength where M_v^* is zero

The ultimate shear strength of a slab where M_v^* is zero, V_{uo} is given by either—

- (a) where there is no shear head—

$$V_{uo} = u d_{om} (f_{cv} + 0.3 \sigma_{cp}) \quad \dots 9.3.3(1)$$

where

$$f_{cv} = 0.17 \left(1 + \frac{2}{\beta_h} \right) \sqrt{f'_c} \leq 0.34 \sqrt{f'_c}; \text{ or}$$

NOTE: The value of σ_{cp} should be evaluated separately for the case of corner, edge and internal columns.

- (b) where there is a shear head—

$$V_{uo} = u d_{om} (0.5 \sqrt{f'_c} + 0.3 \sigma_{cp}) \leq 0.2 u d_{om} f'_c \quad \dots 9.3.3(2)$$

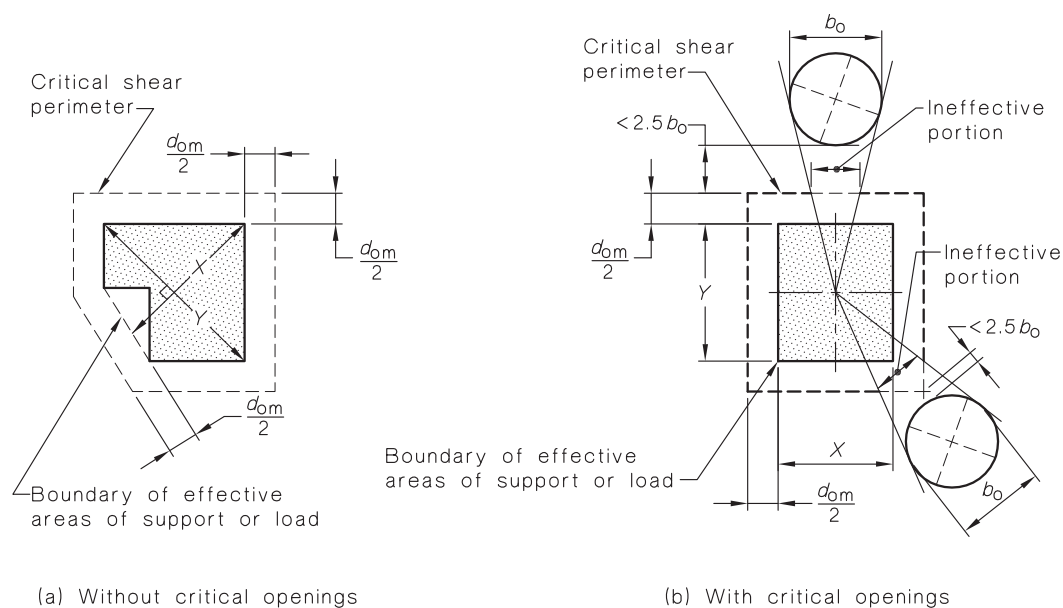


FIGURE 9.3(A) CRITICAL SHEAR PERIMETER

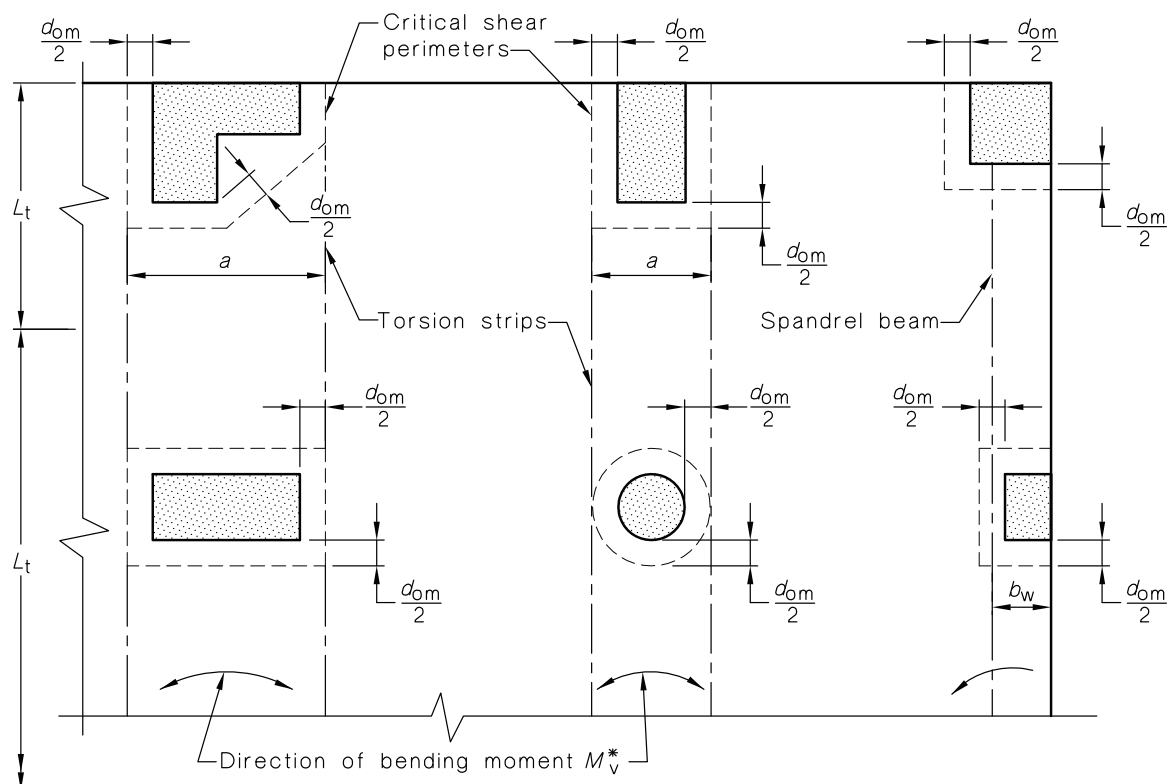


FIGURE 9.3(B) TORSION STRIPS AND SPANDREL BEAMS

9.3.4 Ultimate shear strength where M_v^* is not zero

Where M_v^* is not zero and shear reinforcement, if provided, conforms with Clauses 9.3.5 and 9.2.6, then V_u shall be determined from one of the following:

- (a) If there are no closed fitments in the torsion strip or spandrel beams, V_u is given by—

$$V_u = V_{uo} / [1.0 + uM_v^* / (8V^* ad_{om})] \quad \dots 9.3.4(1)$$

- (b) If the torsion strip contains the minimum quantity of closed fitments, V_u shall be taken as $V_{u,min}$ given by—

$$V_{u,min} = 1.2 V_{uo} / [1.0 + uM_v^* / (2V^* a^2)] \quad \dots 9.3.4(2)$$

- (c) If there are spandrel beams perpendicular to the direction of M_v^* which contain the minimum quantity of closed fitments, V_u shall be taken as $V_{u,min}$ given by—

$$V_{u,min} = 1.2 V_{uo} (D_b / D_s) / [1.0 + uM_v^* / (2V^* ab_w)] \quad \dots 9.3.4(3)$$

- (d) If the torsion strip or spandrel beam contains more than the minimum quantity of closed fitments, V_u is given by—

$$V_u = V_{u,min} \sqrt{[(A_{sw} / s) / (0.2y_1 / f_{sy,f})]} \quad \dots 9.3.4(4)$$

where $V_{u,min}$ is calculated in accordance with Item (b) or (c).

In no case shall V_u be taken greater than $V_{u,max}$ given by—

$$V_{u,max} = 3V_{u,min} \sqrt{(x/y)} \quad \dots 9.3.4(5)$$

where x and y are the shorter and longer dimensions respectively of the cross-section of the torsion strip or spandrel beam.

9.3.5 Minimum area of closed fitments

The minimum cross-sectional area of the reinforcement forming the closed fitments shall satisfy the following inequality:

$$A_{sw} / s \geq 0.2y_1 / f_{sy,f} \quad \dots 9.3.5$$

9.3.6 Detailing of shear reinforcement

Reinforcement for slab shear in torsion strips and spandrel beams shall be in the form of closed fitments arranged and detailed in accordance with the following:

- The fitments shall extend along the torsion strip or spandrel beam for a distance not less than $L_t/4$ from the face of the support or concentrated load, on one or both sides of the centroid axis, as applicable. The first fitment shall be located at not more than $0.5s$ from the face of the support.
- The centre-to-centre spacing (s) of the fitments shall not exceed the lesser of 300 mm and D_b or D_s , as applicable.
- At least one longitudinal bar shall be provided at each corner of the fitment.
- The dimensions of the fitments shall be in accordance with Figure 9.3.6.

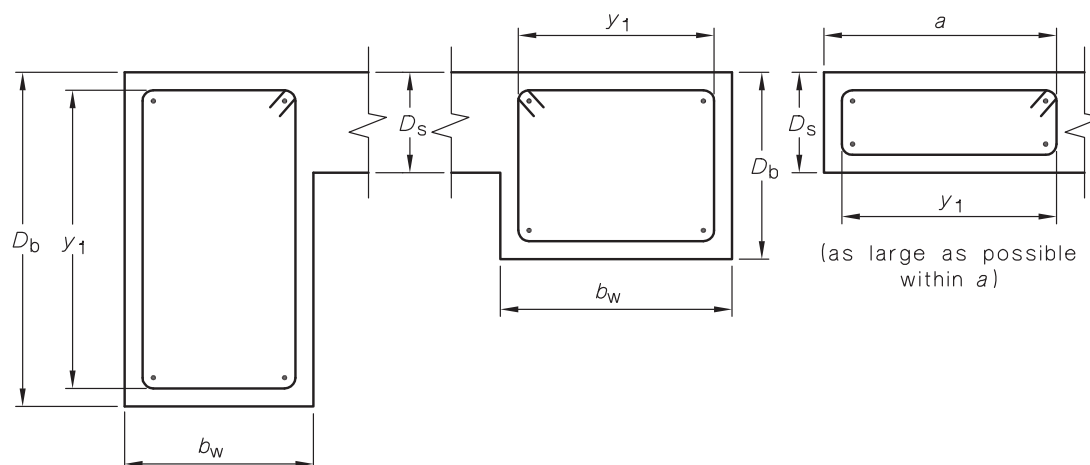


FIGURE 9.3.6 PARAMETERS AND DETAILS OF SHEAR REINFORCEMENT FOR SLABS

9.4 DEFLECTION OF SLABS

9.4.1 General

The deflection of a slab shall be determined in accordance with Clause 9.4.2 or Clause 9.4.3.

Alternatively, for reinforced slabs, the effective span-to-effective depth ratio of the slab shall conform with Clause 9.4.4.

For a slab containing steel fibres in addition to conventional reinforcement or tendons, the deflection shall be determined in accordance with Clause 16.4.7.3.

9.4.2 Slab deflection by refined calculation

The calculation of the deflection of a slab by refined calculation shall make allowance for the following:

- Two-way action.
- Cracking and tension stiffening.
- Shrinkage and creep properties of the concrete.
- Expected load history.
- Expected construction procedure.
- Deflection of formwork or settlement of props during construction, particularly when the slab formwork is supported off suspended floors below.

9.4.3 Slab deflection by simplified calculation

The deflection of a slab subject to uniformly distributed loads shall be calculated in accordance with Clause 8.5.3 on the basis of an equivalent beam taken as follows:

- For a one-way slab, a prismatic beam of unit width.
- For a rectangular slab supported on four sides, a prismatic beam of unit width through the centre of the slab, spanning in the short direction L_x , with the same conditions of continuity as the slab in that direction and with the load distributed so that the proportion of load carried by the beam is given by—

$$L_y^4 / (\alpha L_x^4 + L_y^4) \quad \dots 9.4.3$$

where α is given in Table 9.4.3 for the appropriate slab-edge condition.

- (c) For a two-way flat slab having multiple spans (for deflections on the column lines or midway between the supports), the column strips of the idealized frame described in Clause 6.9.

TABLE 9.4.3
COEFFICIENT OF PROPORTIONALITY (α)

Edge condition		Coefficient (α)
1	Four edges continuous	1.0
2	One short edge discontinuous	0.5
3	One long edge discontinuous	2.0
4	Two short edges discontinuous	0.2
5	Two long edges discontinuous	5.0
6	Two adjacent edges discontinuous	1.0
7	Three edges discontinuous (one long edge continuous)	0.4
8	Three edges discontinuous (one short edge continuous)	2.5
9	Four edges discontinuous	1.0

9.4.4 Deemed to conform span-to-depth ratio for reinforced slabs

9.4.4.1 One-way slabs and two-way flat slabs

For a reinforced one-way slab, or a multiple-span reinforced two-way flat slab of essentially uniform depth, fully propped during construction, subject to uniformly distributed loads and where the imposed action (live load) (q) does not exceed the permanent action (dead load) (g), slab deflections shall be deemed to conform with the requirements of Clause 2.3.2 if the ratio of the effective span to the effective depth satisfies the following:

$$L_{\text{ef}} / d \leq k_3 k_4 \left[\frac{(\Delta / L_{\text{ef}}) 1000 E_c}{F_{\text{d,ef}}} \right]^{1/3} \quad \dots 9.4.4.1$$

NOTE: E_c is in megapascals.

where

Δ / L_{ef} = deflection limit selected in accordance with Clause 2.3.2 and the deflection (Δ) is taken on the centre-line between the supports used to calculate L_{ef}

L_{ef} = effective span

$F_{\text{d,ef}}$ = effective design service load, per unit area, in kilopascals, taken as—

- (a) $(1.0 + k_{\text{cs}})g + (\psi_s + k_{\text{cs}}\psi_1)q$ for total deflection; or
- (b) $k_{\text{cs}}g + (\psi_s + k_{\text{cs}}\psi_1)q$ for the deflection that occurs after the addition or attachment of the brittle partitions or finishes.

k_{cs} is determined in accordance with Clause 8.5.3.2 and ψ_s and ψ_1 are given in AS/NZS 1170.0

k_3 = 1.0 for a one-way slab

= 0.95 for a two-way flat slab without drop panels

= 1.05 for a two-way flat slab with drop panels, which extend at least $L/6$ in each direction on each side of a support centre-line and have an overall depth not less than $1.3D_s$, where D_s is the slab thickness beyond the drops

k_4 = deflection constant, which is taken as—

- (a) for simply supported slabs, 1.4; or
- (b) for continuous slabs, where in adjoining spans the ratio of the longer span to the shorter span does not exceed 1.2 and where no end span is longer than an interior span—
 - (i) 1.75 in an end span; or
 - (ii) 2.1 in interior spans.

9.4.4.2 Rectangular slabs supported on four sides

For a reinforced concrete slab, supported on four sides by walls or beams, subject to uniformly distributed loads and where the imposed action (live load) (q) does not exceed the permanent action (dead load) (g), the slab deflection shall be deemed to conform with the requirements of Clause 2.3.2 if the ratio of the shorter effective span to the effective depth satisfies the requirements given in Clause 9.4.4.1, except that—

- (a) k_3 shall be taken as 1.0; and
- (b) the value of k_4 shall be taken from Table 9.4.4.2.

TABLE 9.4.4.2
SLAB-SYSTEM MULTIPLIER (k_4) FOR RECTANGULAR SLABS
SUPPORTED ON FOUR SIDES

Edge condition	Deflection constant (k_4)			
	Ratio of long to short side (L_y/L_x)			
	1.0	1.25	1.5	2.0
1 Four edges continuous	3.60	3.10	2.80	2.50
2 One short edge discontinuous	3.40	2.90	2.70	2.40
3 One long edge discontinuous	3.40	2.65	2.40	2.10
4 Two short edges discontinuous	3.20	2.80	2.60	2.40
5 Two long edges discontinuous	3.20	2.50	2.00	1.60
6 Two adjacent edges discontinuous	2.95	2.50	2.25	2.00
7 Three edges discontinuous (one long edge continuous)	2.70	2.30	2.20	1.95
8 Three edges discontinuous (one short edge continuous)	2.70	2.10	1.90	1.60
9 Four edges discontinuous	2.25	1.90	1.70	1.50

9.5 CRACK CONTROL OF SLABS

9.5.1 General requirements

Cracking shall be limited to an extent that does not impair the durability or serviceability of the slab, both in terms of function and appearance. A characteristic maximum crack width w'_{\max} shall be selected for each surface of a slab taking into account the function and exposure of the surface.

For regions of slabs fully enclosed within a building except for a brief period of weather exposure during construction, and where it is assessed that cracking will not impair the functioning of the structure, only Items (a) and (b) need be satisfied:

- (a) The minimum area of reinforcement in a tensile zone of a slab shall conform with Clause 9.1.1.

- A2 (b) Unless cracking is deemed to be controlled by prestress in accordance with Clause 9.5.2.3, the centre-to-centre spacing of bars in each direction shall not exceed the lesser of $2.0D_s$ or 300 mm. Bars with a diameter less than half the diameter of the largest bar in the cross-section shall be ignored.

In other situations, in addition to (a) and (b) above, crack widths may be calculated and limited to the selected value of w'_{\max} in accordance with Clause 9.5.2.2. A simplified alternative approach is to limit the size and spacing of reinforcement crossing the cracks in accordance with Clause 9.5.2.1.

NOTES:

- 1 For prestressed elements with unbonded tendons, the requirements for reinforced concrete elements apply. For prestressed elements with bonded tendons, the requirements of Clause 9.5.2.3 apply.
- 2 It is possible that the width of some cracks in the actual structure may exceed the maximum characteristic value specified in design.
- A2 3 Design bending moments at the serviceability limit states will normally be estimated using elastic analysis. Substantial errors may result where the actual in-service moments are likely to have redistributed significantly from the elastic distribution.

9.5.2 Crack control for flexure in reinforced slabs

9.5.2.1 Crack control without direct calculation of crack widths

The calculated tensile steel stress (σ_{scr}) shall not exceed the larger of the maximum steel stresses given in—

- (a) Table 9.5.2.1(A) for the largest nominal diameter (d_b) of the bars in the tensile zone; and
- (b) Table 9.5.2.1(B) for the largest centre-to-centre spacing of adjacent parallel bars in the tensile zone and, when determining spacing, bars with a diameter less than half the diameter of the largest bar in the section shall be ignored.
- A2 (c) The calculated tensile steel stress ($\sigma_{\text{scr},i}$) shall not exceed $0.8f_{\text{sy}}$.

NOTE: [Text deleted]

TABLE 9.5.2.1(A)
MAXIMUM STEEL STRESS FOR FLEXURE IN REINFORCED SLABS

Nominal bar diameter (d_b) mm	Maximum steel stress (MPa) for overall depth, D_s (mm)					
	$w'_{\max} = 0.2$ mm		$w'_{\max} = 0.3$ mm		$w'_{\max} = 0.4$ mm	
	$D_s \leq 300$	$D_s > 300$	$D_s \leq 300$	$D_s > 300$	$D_s \leq 300$	$D_s > 300$
6	210	230	290	315	365	390
8	195	210	260	290	340	360
10	180	190	240	265	315	335
12	165	175	225	245	290	305
16	150	155	205	215	260	270
20	140	140	190	195	235	240
24	125	125	175	175	215	215

TABLE 9.5.2.1(B)
MAXIMUM STEEL STRESS FOR FLEXURE IN REINFORCED SLABS

Centre-to-centre spacing mm	Maximum steel stress (MPa)		
	$w'_{\max} = 0.2 \text{ mm}$	$w'_{\max} = 0.3 \text{ mm}$	$w'_{\max} = 0.4 \text{ mm}$
50	200	300	400
100	170	270	360
150	155	245	330
200	145	225	300
250	135	210	280
300	125	200	260

9.5.2.2 Crack control by calculation of crack widths

The calculated maximum crack width in a reinforced concrete member at any time after cracking shall be determined in accordance with Clause 8.6.2.3.

9.5.2.3 Crack control for flexure in prestressed slabs

Flexural cracking, in a prestressed slab shall be deemed to be controlled if, under the short-term service loads, the resulting maximum tensile stress in the concrete does not exceed $0.25\sqrt{f'_c}$ or, if this stress is exceeded, by providing reinforcement or bonded tendons, or both, near the tensile face with a centre-to-centre spacing not exceeding the lesser of 300 mm or $2.0D_s$ and by one of the following:

- (a) Limiting the calculated maximum flexural tensile stress in the concrete under short-term service loads to $0.6\sqrt{f'_c}$.

or

- (b) Limiting the increment in steel stress near the tension face to that given in Table 9.5.2.3, as the load increases from its value when the extreme concrete tensile fibre is at zero stress to the short-term service load value.

or

- (c) Limiting the calculated maximum crack width in accordance with Clause 8.6.2.2.

TABLE 9.5.2.3
MAXIMUM INCREMENT OF STEEL STRESS FOR FLEXURE
IN PRESTRESSED SLABS

Nominal bar diameter (d_b) mm	Maximum steel stress (MPa) for overall depth, D_s (mm)					
	$w'_{\max} = 0.2 \text{ mm}$		$w'_{\max} = 0.3 \text{ mm}$		$w'_{\max} = 0.4 \text{ mm}$	
	$D_s \leq 300$	$D_s > 300$	$D_s \leq 300$	$D_s > 300$	$D_s \leq 300$	$D_s > 300$
≤ 10	245	280	300	340	350	390
12	210	240	270	300	310	340
16	180	200	240	260	280	300
20	160	170	220	220	255	260
24	145	145	195	195	225	225
≥ 28	120	120	185	185	210	210
All bonded tendons	120	120	185	185	210	210

9.5.3 Crack control for shrinkage and temperature effects

9.5.3.1 General

The area of reinforcement required to control cracking due to shrinkage and temperature effects shall take into account the influence of flexural action, the degree of restraint against in-plane movements and the exposure classification, in accordance with Clauses 9.5.3.2 to 9.5.3.5.

For members greater than 500 mm thick, the reinforcement required near each surface may be calculated using 250 mm for D .

9.5.3.2 Reinforcement in the primary direction

No additional reinforcement is required to control expansion or contraction cracking if the area of reinforcement in the direction of the span of a one-way slab, or in each direction of a two-way slab, is not less than—

- (a) the area required by Clause 9.1.1; and
- (b) 75% of the area required by one of Clauses 9.5.3.3 to 9.5.3.5, as appropriate.

The maximum centre-to-centre spacing of bars required to control cracking due to shrinkage and temperature effects in the primary direction shall conform with Clause 9.5.1(b).

9.5.3.3 Reinforcement in the secondary direction in unrestrained slabs

Where the slab is free to expand or contract in the secondary direction, the minimum area of reinforcement in that direction shall be $(1.75-2.5 \sigma_{cp}) bD \times 10^{-3}$.

9.5.3.4 Reinforcement in the secondary direction in restrained slabs

Where a slab is restrained from expanding or contracting in the secondary direction, the area of reinforcement in that direction shall be not less than the following, as appropriate:

- (a) For a slab fully enclosed within a building except for a brief period of weather exposure during construction:
 - (i) Where a minor degree of control over cracking is required $(1.75-2.5 \sigma_{cp}) bD \times 10^{-3}$.
 - (ii) Where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view $(3.5-2.5 \sigma_{cp}) bD \times 10^{-3}$.
 - (iii) Where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes $(6.0-2.5 \sigma_{cp}) bD \times 10^{-3}$.
- (b) For all other surface and exposure environments in classification A1, and for exposure classification A2:
 - (i) Where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view $(3.5-2.5 \sigma_{cp}) bD \times 10^{-3}$.
 - (ii) Where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes $(6.0-2.5 \sigma_{cp}) bD \times 10^{-3}$.
- (c) For exposure classifications B1, B2, C1 and C2 a strong degree of crack control is always required $(6.0-2.5 \sigma_{cp}) bD \times 10^{-3}$.

The centre-to-centre spacing of bars required to provide a moderate degree of control over cracking shall not exceed the lesser of $2.0D_s$ or 300 mm. For a strong degree of control over cracking, the centre-to-centre spacing of bars shall not exceed the lesser of $1.5D_s$ or 200 mm.

9.5.3.5 Reinforcement in the secondary direction in partially restrained slabs

Where a slab is partially restrained from expanding or contracting in the secondary direction, the minimum area of reinforcement in that direction shall be assessed taking into account the requirements of Clauses 9.5.3.3 and 9.5.3.4.

9.5.4 Crack control in the vicinity of restraints

In the vicinity of restraints, special attention shall be paid to the internal forces and cracks that may be induced by prestressing, shrinkage or temperature.

9.5.5 Crack control at openings and discontinuities

For crack control at openings and discontinuities in a slab, additional properly anchored reinforcement shall be provided if necessary.

9.6 VIBRATION OF SLABS

Vibration in slabs shall be considered and action taken, where necessary, to ensure that the vibrations induced by machinery, or vehicular or pedestrian traffic, will not adversely affect the serviceability of the structure.

9.7 MOMENT RESISTING WIDTH FOR ONE-WAY SLABS SUPPORTING CONCENTRATED LOADS

The width of a solid one-way simply supported or continuous slab deemed to resist the moments caused by a concentrated load, may be taken as follows:

- (a) Where the load is not near an unsupported edge—

$$b_{ef} = \text{the load width} + 2.4a[1.0 - (a/L_n)] \quad \dots 9.7$$

where

a = perpendicular distance from the nearer support to the section under consideration

- (b) Where the load is near an unsupported edge, not greater than the lesser of—

- (i) the value given in Item (a) above; and
- (ii) half the value given in Item (a) above plus the distance from the centre of the load to the unsupported edge.

9.8 LONGITUDINAL SHEAR IN COMPOSITE SLABS

Composite slab systems shall be checked for longitudinal shear at the interfaces between components, in accordance with Clause 8.4.

SECTION 10 DESIGN OF COLUMNS FOR STRENGTH AND SERVICEABILITY

10.1 GENERAL

10.1.1 Design strength

The design strength of a column shall be determined by its ability to resist the axial forces, shear forces and bending moments caused by the design loading for strength and any additional bending moments produced by slenderness effects.

10.1.2 Minimum bending moment

At any cross-section of a column, the design bending moment about each principal axis shall be taken to be not less than N^* times $0.05D$, where D is the overall depth of the column in the plane of the bending moment.

10.1.3 Definitions

For the purpose of this section the definitions below apply.

10.1.3.1 Braced column

Column in a structure for which the lateral actions applied at the ends in the direction under consideration are resisted by components such as masonry infill panels, shear walls or lateral bracing.

10.1.3.2 Short column

Column in which the additional bending moments due to slenderness can be taken as zero.

10.1.3.3 Slender column

Column that does not satisfy the requirements for a short column.

10.2 DESIGN PROCEDURES

10.2.1 Design procedure using linear elastic analysis

Where the axial forces and bending moments are determined by a linear elastic analysis, as provided in Clause 6.2, a column shall be designed as follows:

- (a) For a short column, in accordance with Clauses 10.3, 10.6 and 10.7.
- (b) For a slender column, in accordance with Clauses 10.4 to 10.7.

The value of ϕ shall be determined from Table 2.2.2.

10.2.2 Design procedure incorporating secondary bending moments

Where the axial forces and bending moments are determined by an elastic analysis incorporating secondary bending moments due to lateral joint displacements, as provided in Clause 6.3, a column shall be designed in accordance with Clauses 10.6 and 10.7. The bending moments in slender columns shall be further increased by applying the moment magnifier for a braced column (δ_b) calculated in accordance with Clause 10.4.2 with L_e taken as L_u in the determination of N_c .

The value of ϕ shall be determined from Table 2.2.2.

10.2.3 Design procedure using rigorous analysis

Where the axial forces and bending moments are determined by a rigorous analysis, as provided in Clause 6.5 and 6.6, a column shall be designed in accordance with Clauses 10.6

and 10.7 without further consideration of additional moments due to slenderness. The value of ϕ_{sys} shall be determined from Table 2.2.5.

10.2.4 Design for shear

Design for shear shall be undertaken in accordance with Section 8.2.

Shear reinforcement shall not be less than the fitments required in accordance with Clause 10.7.2 and Section 14 as applicable.

10.3 DESIGN OF SHORT COLUMNS

10.3.1 General

Short columns shall be designed in accordance with this clause and Clauses 10.6 and 10.7, with additional bending moments due to slenderness taken to be zero. Alternatively, for short columns with small axial forces or small bending moments, the design may be in accordance with Clauses 10.3.2 and 10.3.3 respectively.

A column shall be deemed to be short where—

- (a) for a braced column—

$$L_e/r \leq 25; \text{ or}$$

$$\leq \alpha_c (38 - f'_c/15) (1 + M_1^*/M_2^*) \quad \dots 10.3.1(1)$$

whichever is the greater

where

$$\alpha_c = \sqrt{2.25 - 2.5N^*/\phi N_{uo}} \quad \text{for } N^*/\phi N_{uo} \geq 0.15; \text{ or}$$

$$\alpha_c = \sqrt{1/(3.5N^*/\phi N_{uo})} \quad \text{for } N^*/\phi N_{uo} < 0.15$$

- (b) for an unbraced column—

$$L_e/r \leq 22 \quad \dots 10.3.1(2)$$

where, for Items (a) and (b) above—

r = radius of gyration of the cross-sections determined in accordance with Clause 10.5.2

M_1^*/M_2^* = ratio of the smaller to the larger of the design bending moments at the ends of the column.

The ratio is taken to be negative when the column is bent in single curvature and positive when the column is bent in double curvature. When the absolute value of M_2^* is less than or equal to $0.05DN^*$, the ratio shall be taken as -1.0

L_e = effective length determined in accordance with Clause 10.5.3; or alternatively may be taken as—

(i) for a braced column restrained by a flat slab floor, L_u

(ii) for a braced column restrained by beams, $0.9L_u$

ϕ = 0.65

10.3.2 Short column with small compressive axial force

Where the design compressive axial force (N^*) in a short column is less than $0.1f'_cA_g$, the cross-section may be designed for bending only.

10.3.3 Short braced column with small bending moments

The bending moments in a short interior column of a braced rectangular framed building structure may be disregarded if—

- (a) the ratio of the longer to the shorter length of any two adjacent spans does not exceed 1.2;
- (b) the loads are essentially uniformly distributed;
- (c) the imposed action (live load) (q) does not exceed twice the permanent action (dead load) (g);
- (d) members are of uniform cross-section; and
- (e) the cross-section of the column is symmetrically reinforced,

in which case the design axial strength (ϕN_u) is taken as not greater than $0.75\phi N_{uo}$, where N_{uo} is determined in accordance with Clause 10.6.2.2.

10.4 DESIGN OF SLENDER COLUMNS

10.4.1 General

Slender columns shall be designed in accordance with this clause and Clauses 10.5, 10.6 and 10.7, with additional bending moments due to slenderness effects taken into account by multiplying the largest design bending moment by the moment magnifier (δ).

The moment magnifier (δ) shall be calculated in accordance with Clause 10.4.2 for a braced column and Clause 10.4.3 for an unbraced column.

NOTE: The moment magnification factors also apply to the case of minimum moments.

For columns subject to bending about both principal axes, the bending moment about each axis shall be magnified by δ , using the restraint conditions applicable to each plane of bending.

The additional end moments calculated from moment magnification shall be determined by rational calculation, or alternatively, distributed to the members of the joint in proportion to their stiffness.

10.4.2 Moment magnifier for a braced column

The moment magnifier (δ) for a braced column shall be taken to be equal to δ_b given by—

$$\delta_b = k_m / (1 - N^* / N_c) \geq 1 \quad \dots 10.4.2$$

where

N_c = buckling load given in Clause 10.4.4

$k_m = (0.6 - 0.4M_1^* / M_2^*)$ but shall be taken as not less than 0.4, except that if the column is subjected to significant transverse loading between its ends and in the absence of more exact calculations, k_m shall be taken as 1.0

The ratio M_1^* / M_2^* is as defined in Clause 10.3.1.

10.4.3 Moment magnifier for an unbraced column

The moment magnifier (δ) for an unbraced column shall be taken as the larger value of δ_b or δ_s where—

- (a) δ_b for an individual column is calculated in accordance with Clause 10.4.2 assuming the column is braced; and

(b) δ_s for each column in the storey is calculated as—

$$1/(1-\Sigma N^*/\Sigma N_c) \quad \dots 10.4.3(1)$$

where the summations include all columns within the storey and N_c is calculated for each column in accordance with Clause 10.4.4.

As an alternative to Item (b), δ_s may be calculated from a linear elastic critical buckling load analysis of the entire frame, where δ_s is taken as a constant value for all columns given by—

$$\delta_s = 1/[1 - (1 + \beta_d)/(\alpha_s \lambda_{uc})] \quad \dots 10.4.3(2)$$

where

β_d = $G/(G + Q)$ taken as zero when $L_e/r \leq 40$ and $N^* \leq M^*/2D$, and G and Q are the design axial load components due to permanent action (dead load) and imposed action (live load) respectively

α_s = a correlation factor taken as 0.6

λ_{uc} = ratio of the elastic critical buckling load of the entire frame to the design load for strength, calculated by taking the cross-sectional stiffness of the flexural members and columns as $0.4E_c I_f$ and $0.8E_c I_c$ respectively

The frame shall be proportioned so that δ_s for any column is not greater than 1.5.

10.4.4 Buckling load

The buckling load (N_c) shall be taken as—

$$N_c = (\pi^2 / L_e^2) [182 d_o (\phi M_c) / (1 + \beta_d)] \quad \dots 10.4.4$$

where

M_c = M_{ub} with $k_u = 0.545$ and $\phi = 0.65$

10.5 SLENDERNESS

10.5.1 General

The slenderness ratio (L_e/r) of a column shall not exceed 120, unless a rigorous analysis has been carried out in accordance with Clauses 6.4, 6.5 or 6.6 and the column is designed in accordance with Clause 10.2.3.

Where the forces and moments acting on a column have been obtained from a linear elastic analysis, as specified in Clause 6.2, the influence of slenderness shall be taken into account using a radius of gyration (r) specified in Clause 10.5.2 and an effective length (L_e), in accordance with Clause 10.5.3.

10.5.2 Radius of gyration

The radius of gyration (r) shall be calculated for the gross concrete cross-section. For a rectangular cross-section, r shall be taken as $0.3D$, where D is the overall dimension in the direction in which stability is being considered and for a circular cross-section, r shall be taken as $0.25D$.

10.5.3 Effective length of a column

The effective length of a column (L_e) shall be taken as kL_u where the effective length factor (k) is determined from Figure 10.5.3(A) for columns with simple end restraints, or more generally from Figures 10.5.3(B) or 10.5.3(C).

The end restraint coefficients (γ_1 and γ_2) shall be determined—

- (a) for regular rectangular framed structures where the axial forces in the beams are generally small, in accordance with Clause 10.5.4;
- (b) for all structures, including non-rectangular framed structures or structures where the axial forces in the restraining members are large, in accordance with Clause 10.5.5; and
- (c) where the column ends at a footing, in accordance with Clause 10.5.6.

Alternatively, the effective length of a column may be determined from the elastic critical buckling load of the frame, as calculated by analysis.

	Braced column			Unbraced column		
Buckled shape						
Effective length factor (k)	0.70	0.85	1.00	1.20	2.20	2.20
Symbols for end restraint conditions	= Rotation fixed, translation fixed			= Rotation fixed, translation free		
	= Rotation free, translation fixed			= Rotation free, translation free		

FIGURE 10.5.3(A) EFFECTIVE LENGTH FACTOR (k) FOR COLUMNS WITH SIMPLE END RESTRAINTS

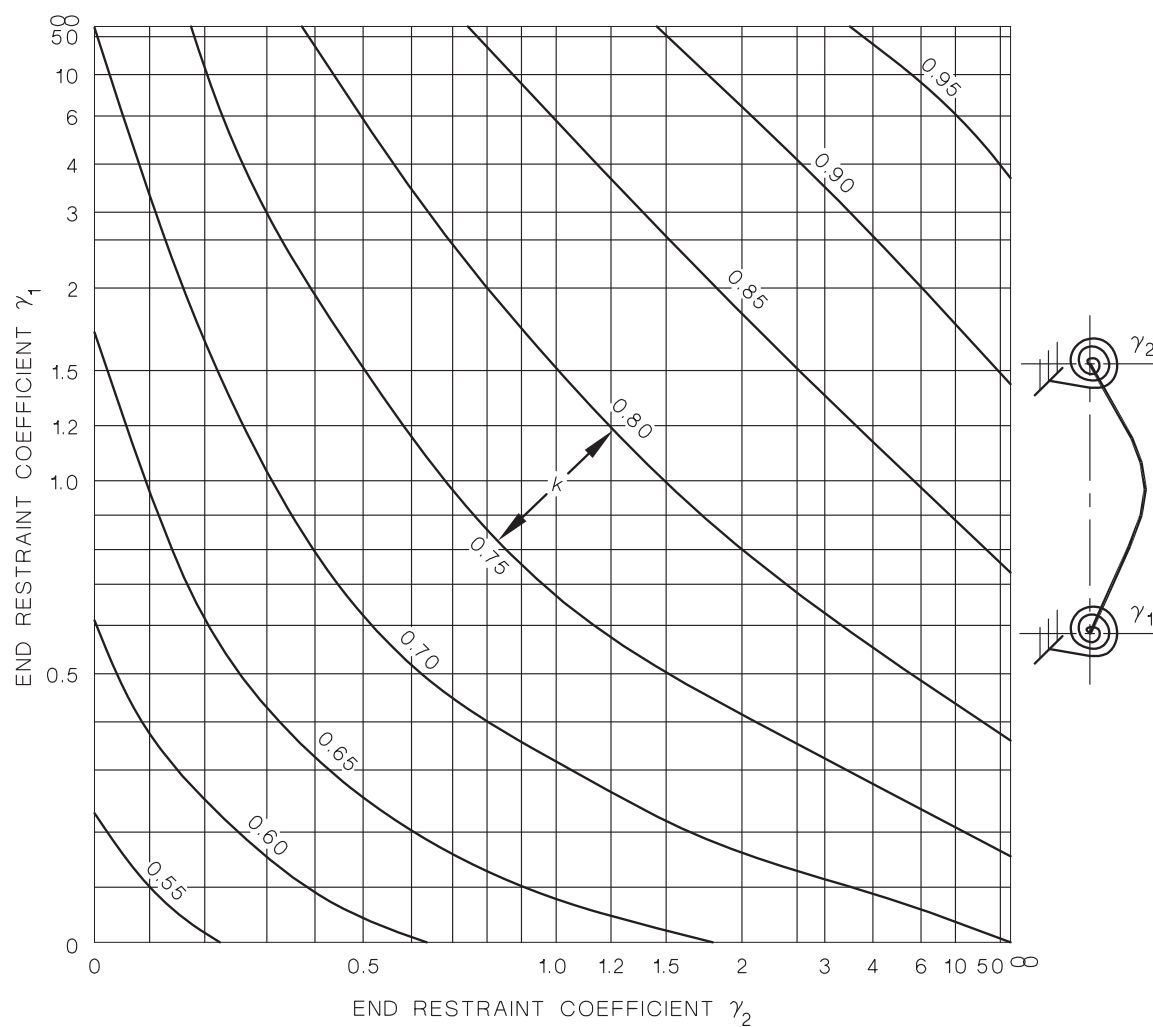
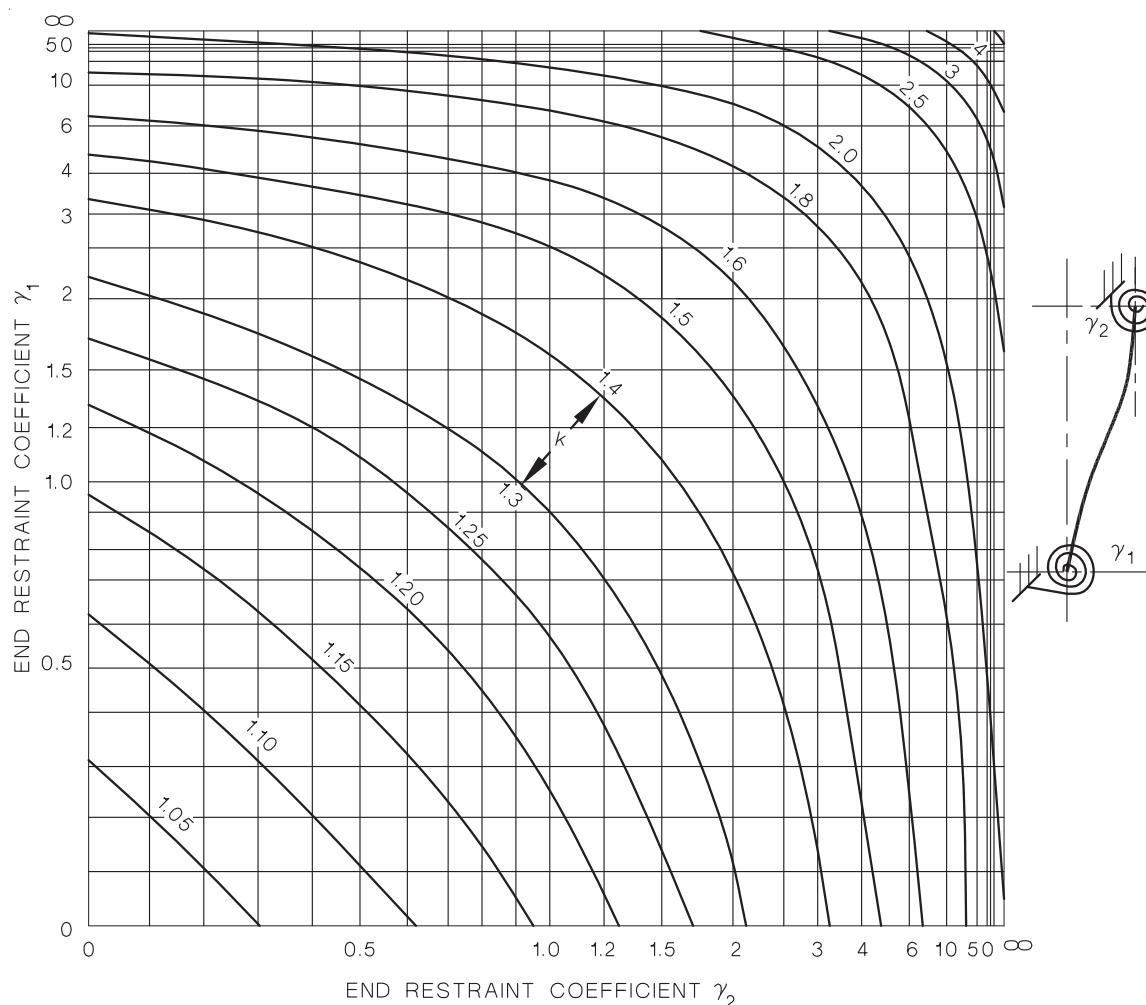


FIGURE 10.5.3(B) EFFECTIVE LENGTH FACTOR (k) FOR BRACED COLUMNS

FIGURE 10.5.3(C) EFFECTIVE LENGTH FACTOR (k) FOR UNBRACED COLUMNS

10.5.4 End restraint coefficients for regular rectangular framed structures

For regular rectangular framed structures, the end restraint coefficient (γ_1) at one end of a column and the end restraint coefficient (γ_2) at the other end shall be determined by rational calculation, alternatively each may be calculated as—

$$\frac{\sum(I/L)_c}{\sum(\beta I/L)_b} \quad \dots 10.5.4$$

where

$\sum(I/L)_c$ = sum of the stiffness in the plane of bending of all the columns meeting at and rigidly connected to the end of the column under consideration

$\sum(\beta I/L)_b$ = sum of the stiffness in the plane of bending of all the beams or slabs, or both, meeting at and rigidly connected to the same end of the column under consideration

β = a fixity factor, given in Table 10.5.4, for fixity conditions at the end of each beam or slab, or both, opposite to the end connected to the column under consideration

TABLE 10.5.4
FIXITY FACTOR (β)

Fixity conditions at far end of a beam or slab, or both	Fixity factor (β)	
	Beam or slab or both, in a braced frame	Beam or slab or both, in an unbraced frame
Pinned	1.5	0.5
Rigidly connected to a column	1.0	1.0
Fixed	2.0	0.67

10.5.5 End restraint coefficients for any framed structure

For any framed structure, the end restraint coefficient (γ_1) at one end of a column and the end restraint coefficient (γ_2) at the other end may be calculated as the ratio of the column stiffness to the sum of the stiffnesses of all the members, except the column, meeting at the end under consideration. In the calculation of the stiffness of members, other than the column, due account shall be taken of the fixity conditions of each member at the end remote from the column-end being considered as well as any reduction in member stiffness due to axial compression.

10.5.6 End restraint provided by footings

Where a footing provides negligible restraint to the rotation of the end of a column, γ is theoretically infinite but may be taken as 10.

Where a footing is specifically designed to prevent rotation of the end of a column, γ is theoretically zero but shall be taken as 1.0 unless analysis would justify a smaller value.

10.6 STRENGTH OF COLUMNS IN COMBINED BENDING AND COMPRESSION

10.6.1 Basis of strength calculations

Calculations for the strength of cross-sections in bending, combined with axial forces, shall incorporate equilibrium and strain-compatibility considerations and be consistent with the following assumptions:

- Plane sections normal to the axis remain plane after bending.
- The concrete has no tensile strength.
- The distribution of stress in the concrete and the steel is determined using a stress-strain relationship determined from Clauses 3.1.4 and 3.2.3 respectively (see Note 1).
- The strain in compressive reinforcement does not exceed 0.003.
- Where the neutral axis lies outside of the cross-section, consideration shall be given to the effect on strength of spalling of the cover concrete.

NOTES:

- If a curvilinear stress-strain relationship is used then—
 - Clause 3.1.4 places a limit on the value of the maximum concrete stress; and
 - the strain in the extreme fibre may be adjusted to obtain the maximum bending strength for a given axial load.
- The effect of confinement on the strength of a section may be taken into account, provided secondary effects such as concrete spalling, for example, are also considered.

Columns subject to axial force with bending moments about each principal axis may take into account the concessions given in Clauses 10.6.3 and 10.6.4.

10.6.2 Strength of cross-sections calculated using the rectangular stress block

10.6.2.1 General

It shall be permissible to represent the strength of a cross-section in combined bending and compression using a strength interaction diagram as shown in Figure 10.6.2.1 defined as given in Clauses 10.6.2.2 to 10.6.2.5.

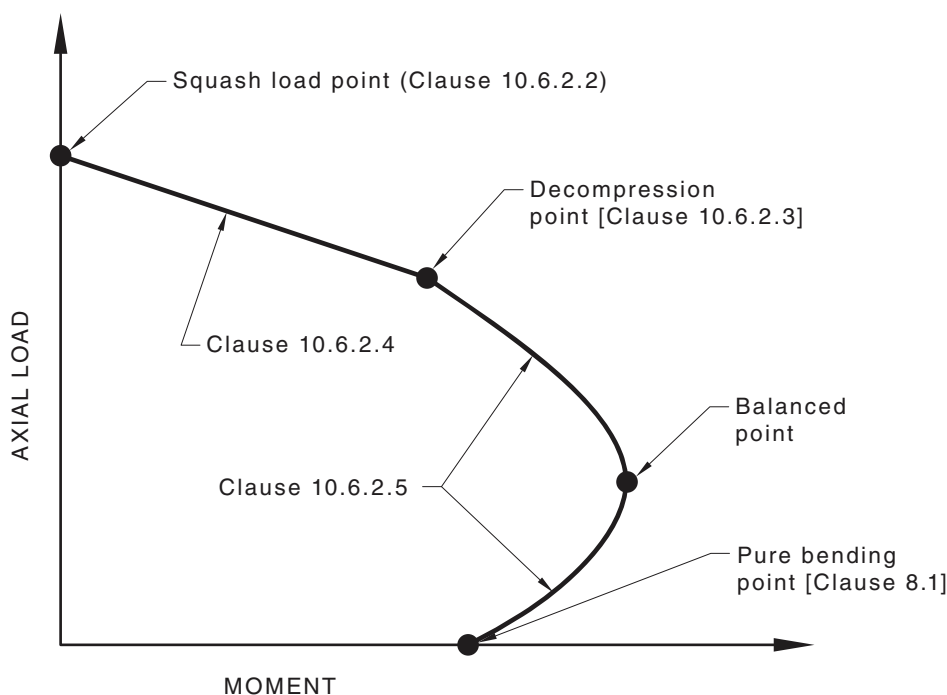


FIGURE 10.6.2.1 AXIAL LOAD—MOMENT DIAGRAM

10.6.2.2 Squash load (N_{uo})

The ultimate strength in compression without bending (N_{uo}) shall be calculated by assuming—

- (a) a uniform concrete compressive stress of $\alpha_1 f'_c$, where—

$$\alpha_1 = 1.0 - 0.003 f'_c \text{ with the limits } 0.72 \text{ to } 0.85; \text{ and} \quad \dots 10.6.2.2$$

NOTE: The modification of $0.9 f'_c$ given in Clause 3.1.4 is included in the rectangular stress block assumptions.

- (b) a maximum strain in the reinforcement of 0.0025.

10.6.2.3 Decompression point

The decompression point is calculated taking the strain in the extreme compressive fibre equal to 0.003, the strain in the extreme tensile fibre equal to zero and using the rectangular stress block given in Clause 10.6.2.5.

10.6.2.4 Transition from decompression point to squash load

Where the neutral axis lies outside of the section, the section strength may be calculated using a linear relationship between the decompression point given by Clause 10.6.2.3 and the squash load (N_{uo}) calculated using Clause 10.6.2.2.

10.6.2.5 Transition from decompression point to bending strength

Where the neutral axis lies within the cross-section and provided the maximum strain in the extreme compression fibre of the concrete is taken as 0.003, Clause 10.6.1(c) shall be deemed to be satisfied for the concrete by assuming a uniform concrete compressive stress of $\alpha_2 f'_c$ acts on an area bounded by—

- (a) the edges of the cross-section; and
- (b) a line parallel to the neutral axis under the loading concerned, and located at a distance $\gamma k_u d$ from the extreme compressive fibre, where—

$$\alpha_2 = 0.85 - 0.0015 f'_c \quad \alpha_2 \geq 0.67 \quad \dots 10.6.2.5(1)$$

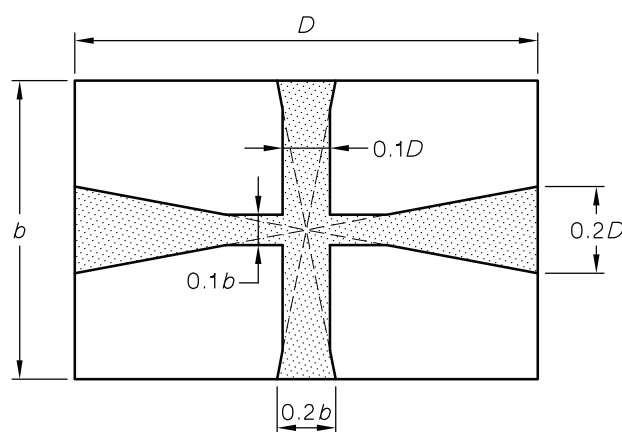
$$\gamma = 0.97 - 0.0025 f'_c \quad \gamma \geq 0.67 \quad \dots 10.6.2.5(2)$$

NOTES:

- 1 The modification of $0.9 f'_c$ given in Clause 3.1.4 is included in the rectangular stress block assumptions.
- 2 Cover spalling may be a problem in columns cast with high-strength concrete. The effect of cover spalling on strength given in Clause 10.6.1(e) is included in the parameters developed for the calculation of the interaction diagram.
- 3 For circular sections α_2 shall be reduced by 5 percent and for any other sections where the width reduces from the neutral axis towards the compression face, α_2 shall be reduced by 10 percent.

10.6.3 Design based on each bending moment acting separately

For a rectangular cross-section, where the ratio of the larger to the smaller cross-sectional dimension does not exceed 3.0, which is subjected simultaneously to an axial force and bending moments about each principal axis, the cross-section may be designed for the axial force with each bending moment considered separately, provided the line of action of the resultant force falls within the shaded area of the cross-section shown in Figure 10.6.3.



Shaded areas symmetrical about column centre-lines

FIGURE 10.6.3 LIMITATION FOR LINE OF ACTION OF THE RESULTANT AXIAL FORCE

10.6.4 Design for biaxial bending and compression

A rectangular cross-section, subject to axial force and bending moment acting simultaneously about each principal axis shall be determined by rational calculation, alternatively it may be designed such that—

$$\left(\frac{M_x^*}{\phi M_{ux}} \right)^{\alpha_n} + \left(\frac{M_y^*}{\phi M_{uy}} \right)^{\alpha_n} \leq 1.0 \quad \dots 10.6.4$$

where

M_{ux}, M_{uy} = strength in bending, calculated separately, about the major and minor axis respectively under the design axial force (N^*)

M_x^*, M_y^* = design bending moment about the major and minor axis respectively, magnified, if applicable

α_n = $0.7 + 1.7N^*/\phi N_{uo}$, within the limits $1 \leq \alpha_n \leq 2$

ϕ = 0.65

10.7 REINFORCEMENT REQUIREMENTS FOR COLUMNS

10.7.1 Limitations on longitudinal steel

The cross-sectional area of the longitudinal reinforcement in a column shall—

- (a) be not less than $0.01A_g$ except that, in a column that has a larger area than that required for strength, a reduced value of A_{sc} may be used if $A_{sc}f_{sy} > 0.15N^*$; and
- (b) not exceed $0.04A_g$ unless the amount and disposition of the reinforcement will not prevent the proper placing and compaction of the concrete at splices and at junctions of the members.

Groups of parallel longitudinal bars, that are bundled to act as a unit, shall have not more than four bars in any one bundle and shall be tied together in contact.

10.7.2 Functions of fitments

In addition to meeting the shear and torsion requirements of Clauses 8.2 and 8.3, fitments shall satisfy the requirements of confinement of concrete (Clause 10.7.3), lateral restraint of longitudinal bars against premature buckling (Clause 10.7.4) and confinement of the concrete for earthquake actions as set out in Section 14 for moment resisting frames.

10.7.3 Confinement to the core

10.7.3.1 General requirements

Fitments (including helical reinforcement) shall be detailed to provide confinement to the core of the column—

- (a) for columns where $f'_c \leq 50$ MPa confinement shall be deemed to be provided if the requirements of Clause 10.7.4 are satisfied; and
- (b) for columns where $f'_c > 50$ MPa confinement shall be provided—
 - (i) in special confinement regions using fitments detailed to provide a minimum effective confining pressure to the core of $0.01 f'_c$, calculated in accordance with Clauses 10.7.3.2, 10.7.3.3 or 10.7.3.4; and
 - (ii) outside of the special confinement regions, confinement shall be deemed to be provided if the maximum spacing of the fitments does not exceed the lesser of $0.8b$, 300 mm and that of Clause 10.7.4.

- A2 | In the special confinement regions, the spacing or pitch of the fitments shall not exceed the lesser of $0.6 b D_c$, 300 mm and that of Clause 10.7.4.
- A2 | NOTE: See also Clauses 14.4.3, 14.5.4 and 14.5.5 for additional provisions for members subject to earthquake actions.

Special confinement regions are regions where the design action effects satisfy one of the following [see Figure 10.7.3.1(A)]:

- (A) $N^* \geq 0.75 \phi N_{uo}$. . . 10.7.3.1(1)
- (B) $N^* \geq \phi 0.3 f'_c A_g$ and $M^* \geq 0.6 \phi M_u$. . . 10.7.3.1(2)

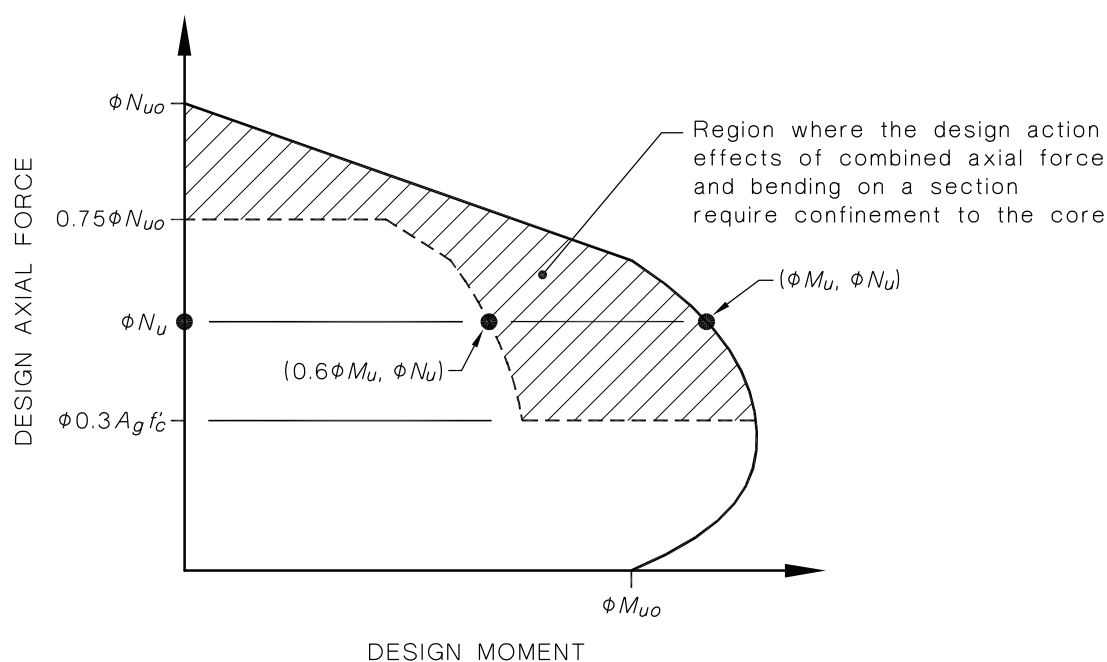
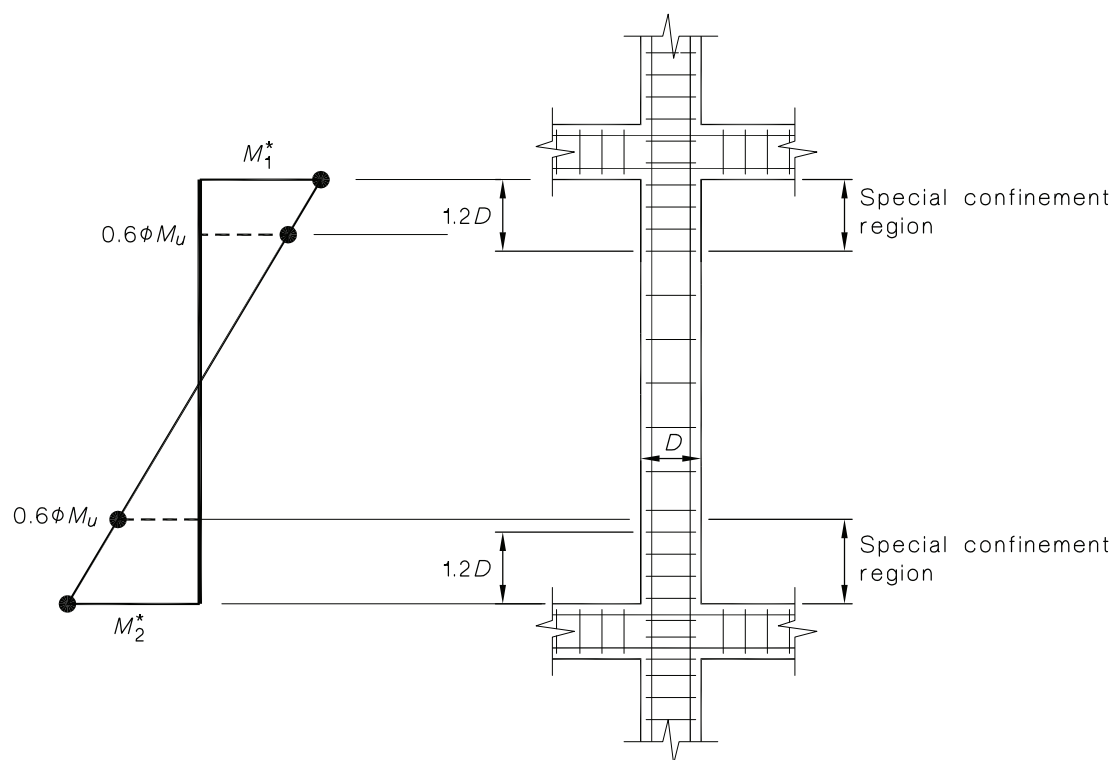


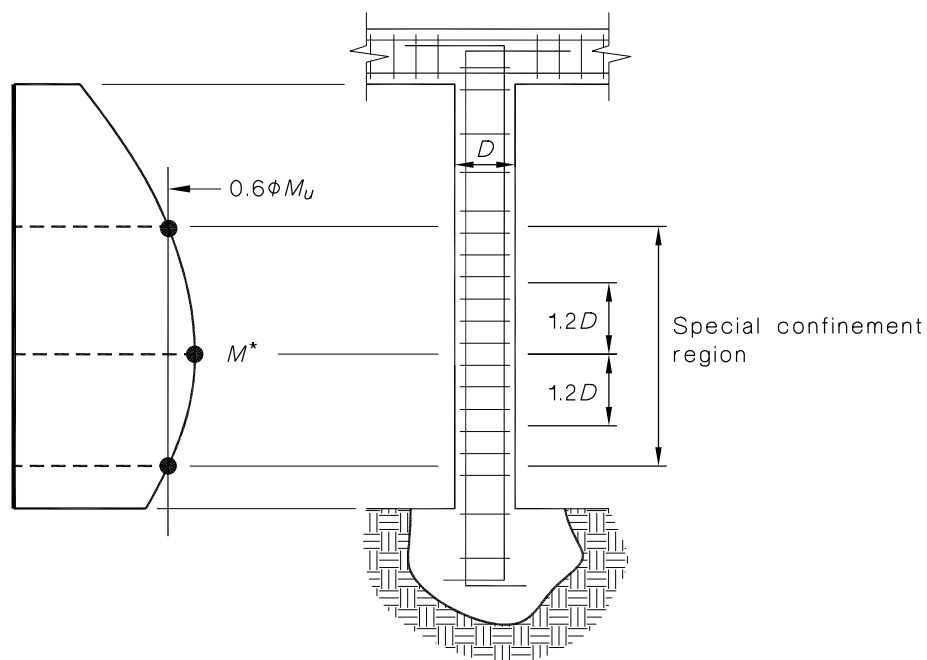
FIGURE 10.7.3.1(A) CONFINEMENT TO THE CORE

Fitments in special confinement regions shall be provided over the limits defined by the special confinement regions [see Figure 10.7.3.1(B)], but not less than a length measured each side of the maximum moment and bounded by the lesser of—

- (1) 1.2 times the dimension of the cross-section measured normal to the axis of bending being considered; and
- (2) the distance to the end of the member.



(a) Column with confinement regions at member ends for $N^* \leq 0.75\phi N_{uo}$



(b) Column with confinement region within member length for $N^* \leq 0.75\phi N_{uo}$

FIGURE 10.7.3.1(B) SPECIAL CONFINEMENT REGIONS

For columns in moment resisting frame structures that are in double curvature where $0.3\phi f'_c A_g < N^* < 0.75\phi N_{uo}$, the special confinement regions may be taken as the length from each end of the member equal to the larger of—

$$(aa) \quad 0.4L \frac{|M_2^*|}{|M_1^*| + |M_2^*|} \quad \dots 10.7.3.1(3)$$

(bb) 1.2 times that of the larger cross-sectional dimension.

10.7.3.2 Calculation of core confinement by rational calculation

The confining pressure applied to the core of a column shall be obtained with consideration of triaxial stresses across all sections and the effectiveness of fitments in providing the specified confinement.

10.7.3.3 Calculation of core confinement by simplified calculation

The effective confining pressure applied to the core of a column is calculated as—

$$f_{r,eff} = k_e f_r \quad \dots 10.7.3.3(1)$$

where

k_e = an effectiveness factor accounting for the arrangement of the fitments

f_r = average confining pressure on the core cross-section taken at the level of the fitments (see Figure 10.7.3.3)

NOTE: For non-circular sections, f_r is taken as the smaller of the confining pressures calculated for each of the major directions [XX and YY shown in Figure 10.7.3.3(d)].

The average confining pressure on the core at the level of the fitments shall be calculated as follows:

$$f_r = \frac{\sum_{i=1}^m A_{b,fit} f_{sy,f} \sin \theta}{d_s s} \quad \dots 10.7.3.3(2)$$

where

$A_{b,fit}$ = cross-sectional area of one leg of the fitment

$f_{sy,f}$ = yield stress of the reinforcement used as fitment (not greater than 800 MPa)

θ = angle between the fitment leg and the confinement plane

m = number of fitment legs crossing the confinement plane

d_s = overall dimension measured between centre-lines of the outermost fitments

s = centre to centre spacing of fitments along the column

The effectiveness factor shall be calculated as follows:

(a) For rectangular sections

$$k_e = \left(1 - \frac{nw^2}{6A_c}\right) \left(1 - \frac{s}{2b_c}\right) \left(1 - \frac{s}{2d_c}\right) \quad \dots 10.7.3.3(3)$$

where

A_c = cross-sectional area bounded by the centre-line of the outermost fitments

n = number of laterally restrained longitudinal bars [see Clause 10.7.4.2(a)]

w = average clear spacing between adjacent restrained longitudinal bars

b_c = core dimension measured between the centre-lines of the outermost fitments measured across the width of the section

d_c = core dimension measured between the centre-lines of the outermost fitments measured through the depth of the section

(b) For circular sections

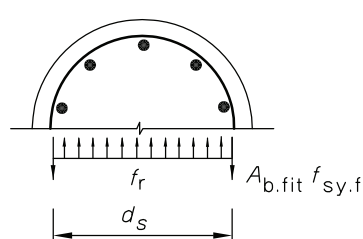
$$k_e = \left(1 - \frac{s}{2d_s}\right)^2 \quad \dots 10.7.3.3(4)$$

Alternatively, for rectangular or circular columns, the effective confining pressure may be calculated as follows:

$$f_{r,eff} = 0.5k_e\rho_s f_{sy,f} \quad \dots 10.7.3.3(5)$$

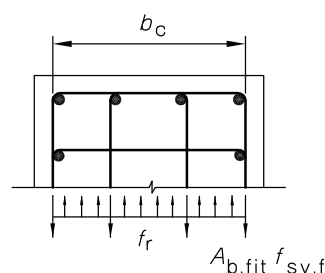
where

ρ_s = the volumetric ratio of the fitments relative to the volume of the core
calculated as $\rho_s = \frac{A_{b,fit} \times \text{total perimeter of fitments crossing the section}}{A_c \times s}$



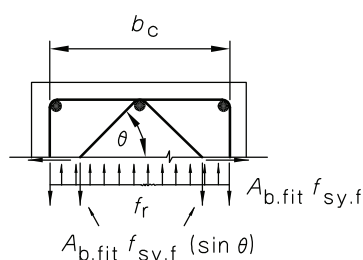
$$f_r = \frac{2A_{b,fit} f_{sy,f}}{d_s s}$$

(a)



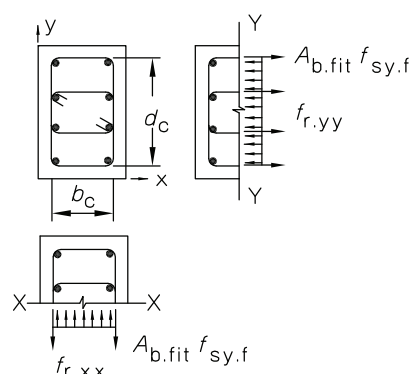
$$f_r = \frac{4A_{b,fit} f_{sy,f}}{b_c s}$$

(b)



$$f_r = \frac{2A_{b,fit} f_{sy,f} (1 + \sin \theta)}{b_c s}$$

(c)



$$f_r = \min(f_{r,xx}, f_{r,yy})$$

(d)

FIGURE 10.7.3.3 CALCULATION OF CONFINING PRESSURES

10.7.3.4 Deemed to conform core confinement

The requirement of an effective confining pressure of $0.01f'_c$ is deemed to be satisfied provided the fitment spacings do not exceed the following:

- (a) For rectangular sections:

$$\frac{15nA_{b,fit}f_{sy,f}}{f'_c\sqrt{A_c}} \quad \dots 10.7.3.4(1)$$

where

n = number of laterally restrained longitudinal bars [see Clause 10.7.4.2(a)]

- (b) For circular sections:

$$\frac{100A_{b,fit}f_{sy,f}}{d_s f'_c} \quad \dots 10.7.3.4(2)$$

10.7.4 Restraint of longitudinal reinforcement

10.7.4.1 General requirements

The following longitudinal bars in columns shall be laterally restrained in accordance with Clause 10.7.4.2:

- (a) Single bars—
- (i) each corner bar;
 - (ii) all bars, where bars are spaced at centres of more than 150 mm or where the design axial force $> 0.3A_g f'_c$; and
 - (iii) at least every alternate bar, where bars are spaced at 150 mm or less.
- (b) Bundled bars—each bundle.

10.7.4.2 Lateral restraint

Lateral restraint shall be deemed to be provided if the longitudinal reinforcement is placed within and in contact with—

- (a) a non-circular fitment (see Figure 10.7.4.2)—
- (i) at a bend in the fitment, where the bend has an included angle of 135° or less; or
 - (ii) between two 135° fitment hooks; or
 - (iii) inside a single 135° fitment hook of a fitment that is approximately perpendicular to the column face; or
 - (iv) for internal fitments with one leg, at a 90° fitment hook of a fitment that is approximately perpendicular to the column face, provided—
 - (A) the other end of the fitment has a 135° hook around a longitudinal reinforcing bar;
 - (B) the end types on adjacent fitments in plan are alternated;
 - (C) the end types on consecutive internal fitments along the longitudinal reinforcement are alternated;
 - (D) the design axial force is $\leq 0.3A_g f'_c$; and
 - (E) $f'_c \leq 65$ MPa.

- (b) a circular fitment or helical reinforcement and the longitudinal reinforcing bars are equally spaced around the circumference.

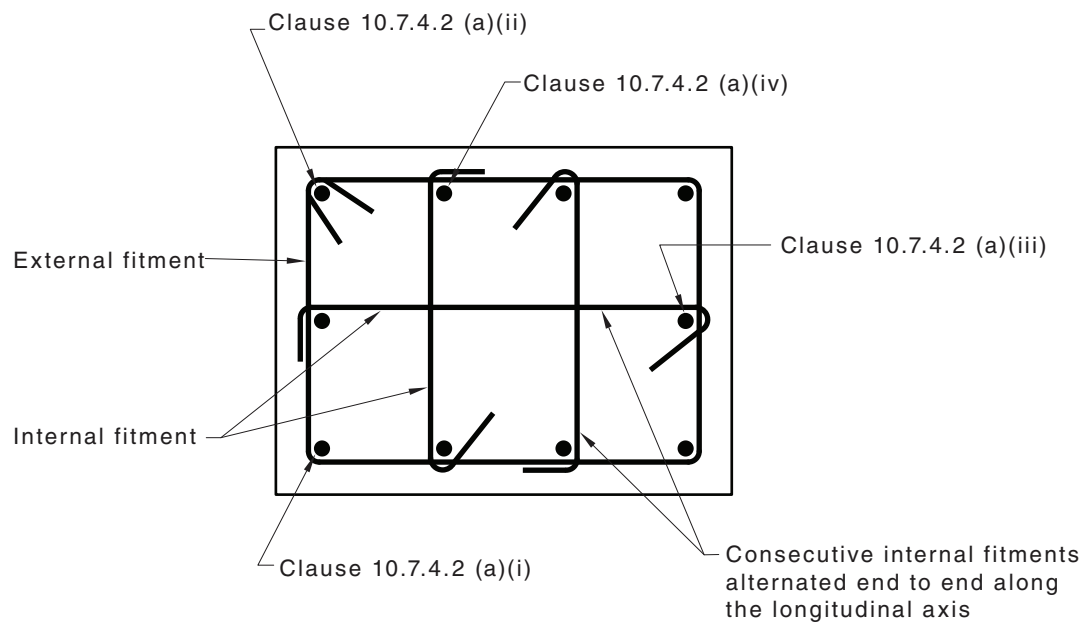


FIGURE 10.7.4.2 LATERAL RESTRAINT TO LONGITUDINAL BARS

10.7.4.3 Diameter and spacing of fitments and helices

The diameter and spacing of fitments and helices shall conform with the following:

- (a) The bar diameter of the fitment and helical reinforcement shall be not less than that given in Table 10.7.4.3.

TABLE 10.7.4.3
BAR DIAMETERS FOR FITMENTS AND HELICES

Longitudinal bar diameter mm	Minimum bar diameter of fitment and helix mm
Single bars up to 20	6
Single bars 24 to 28	10
Single bars 32 to 36	12
Single bars ≥ 40	16
Bundled bars	12

NOTE: For fitments of strength greater than 500 MPa, the minimum bar diameter of the fitment may be reduced by the factor $\sqrt{500 / f_{sy,f}}$.

- (b) The spacing of fitments, or the pitch of a helix, shall not exceed the smaller of—

- (i) b and $15d_b$ for single bars; or
(ii) $0.5b$ and $7.5d_b$ for bundled bars;

where

b = smaller column cross-sectional dimension if rectangular or the column diameter if circular

d_b = diameter of the smallest bar in the column

(iii) where $L_u \leq 5D$ the spacing of the fitments shall not exceed the requirements of Section 14.5.

- (c) One fitment, or the first turn of a helix, shall be located not more than 50 mm vertically above the top of a footing, or the top of a slab in any storey. Another fitment, or the final turn of a helix, shall be located not more than 50 mm vertically below the soffit of the slab except that in a column with a capital, the fitment or turn of the helical reinforcement shall be located at a level at which the area of the cross-section of the capital is not less than twice that of the column.

Where beams or brackets frame from four directions into a column and restrain the column in all directions, the fitments or helical reinforcement may be terminated 50 mm below the highest soffit of such beams or brackets.

NOTE: Welded wire mesh, having strength and anchorage equivalent to that required for bars, may be used.

10.7.4.4 Detailing of fitments and helices

Detailing of fitments and helices shall be as follows:

- (a) A rectangular fitment shall be spliced by welding, or by fixing two 135° fitment hooks around a bar or a bundle at a fitment corner. Internal fitments may be spliced by lapping within the column core.
- (b) A circular shaped fitment shall be spliced either by welding, or by overlapping and fixing two 135° fitment hooks around adjacent longitudinal bars or bundles.
- (c) A helical reinforcement shall be anchored at its end by one and one half extra turns of the helix. It may be spliced within its length either by welding or mechanical means.
- (d) Where hooks or cogs are specified in combination with bundled bars, the internal diameter of the bend shall be increased readily accommodate the bundle.

10.7.4.5 Column joint reinforcement

Where bending moments from a floor system are transferred to a column, lateral shear reinforcement, of area $A_{sv} \geq 0.35 bs/f_{sy.f}$, shall be provided through the joint except that where a slab or beams exist on all four sides of the column, the fitments may be omitted over the depth of the shallowest slab or beam. The spacing of fitments shall be in accordance with Clause 10.7.4.3.

10.7.5 Splicing of longitudinal reinforcement

10.7.5.1 General

Longitudinal reinforcement in columns shall be spliced in accordance with Clauses 10.7.5.2 to 10.7.5.5. The splices shall conform with Clause 13.2.

10.7.5.2 Minimum tensile strength

At any splice in a column, a tensile strength in each face of the column of not less than $0.25f_{sy}A_s$ shall be provided, where A_s is the cross-sectional area of longitudinal reinforcement in that face.

10.7.5.3 Where tensile force exceeds the minimum tensile strength

At any splice in a column where tensile stress exists and the tensile force in the longitudinal bars at any face of the column, due to strength design load effects, exceeds the minimum strength requirements given in Clause 10.7.5.2, the force in the bars shall be transmitted by—

- (a) a welded or mechanical splice in accordance with Clause 13.2.6; or
- (b) a lap-splice in tension in accordance with Clause 13.2.2 or 13.2.5.

10.7.5.4 End-bearing splice in compression

Where the splice is always in compression, the force in the longitudinal bar may be transmitted by the bearing of square-cut mating ends held in concentric contact by a sleeve, provided an additional fitment, which conforms with Clause 10.7.4, is placed above and below each sleeve. The bars shall be rotated to achieve the maximum possible area of contact between the ends of the bars and the requirements of Clause 10.7.5.2 shall be met.

10.7.5.5 Offset bars

Where a longitudinal bar is offset to form a lap splice—

- (a) the slope of the inclined part of the bar in relation to the axis shall not exceed one in six;
- (b) the portions of the bar on either side of the offset shall be parallel; and
- (c) designed lateral support shall be provided at the offset.

Where a column face is offset 75 mm or greater, longitudinal bars shall not be offset by bending but shall be lap-spliced with separate splicing bars placed adjacent to the offset column faces.

10.8 TRANSMISSION OF AXIAL FORCE THROUGH FLOOR SYSTEMS

The transmission of axial force through floor systems shall be designed in accordance with the following:

- (a) If the concrete strength specified for the floor system is greater than or equal to 0.75 times that specified for the column and the longitudinal reinforcement is continuous through the joint, transmission of forces through the joint is deemed to be provided.
- (b) If the concrete strength specified for the floor system is less than 0.75 times that specified for the column, transmission of force through the floor concrete shall be provided by adding longitudinal reinforcement through the joint, in accordance with calculations made using the effective compressive strength of the concrete in the joint (f'_{ce}) as follows:
 - (i) For columns restrained on four sides by beams of approximately equal depth or by a slab, f'_{ce} shall be taken as—

$$f'_{ce} = \left[1.33 - \frac{0.33}{h/b} \right] f'_{cs} + \frac{0.25}{h/b} f'_{cc} \quad \text{within the limits} \quad \dots 10.8(1)$$

$$\min(f'_{cc}, 1.33f'_{cs}) \leq f'_{ce} \leq \min(f'_{cc}, 2.5f'_{cs})$$

where

h = overall depth of the joint

b = smaller column cross-sectional dimension

f'_{cc} = compressive strength of the concrete in the column

f'_{cs} = compressive strength of the concrete in the slab or beams

The value of h/b shall not be taken as less than 0.33.

- (ii) For columns restrained on two opposing sides by beams of approximately equal depth or by a slab, f'_{ce} shall be taken as—

$$f'_{ce} = \left[1.1 - \frac{0.3}{h/b} \right] f'_{cs} + \frac{0.2}{h/b} f'_{cc} \quad \text{within the limits} \quad \dots 10.8.(2)$$

$$\min(f'_{cc}, 1.33f'_{cs}) \leq f'_{ce} \leq \min(f'_{cc}, 2.0f'_{cs})$$

A2 | The value of h/b shall not be taken as less than 0.33.

- (iii) For columns restrained on two adjacent sides by beams of approximately equal depth or by a slab, f'_{ce} shall be taken as—

$$f'_{ce} = 1.33f'_{cs} \leq f'_{cc} \quad \dots 10.8.(3)$$

NOTES:

- 1 Confining reinforcement may be used to increase the effective strength of the concrete in the joint.
- 2 Beams and slabs should provide restraint over the width of the column joint.

10.9 CRACK CONTROL

For the control of flexural cracking in a column, the requirements of Clause 8.6 shall be satisfied.

SECTION 11 DESIGN OF WALLS

11.1 GENERAL

This section applies to the following:

- (a) Braced walls (as defined in Clause 11.3) that are subject to in-plane load effects, which shall be designed in accordance with Clauses 11.2 to 11.7.
- (b) Braced walls that are subject to simultaneous in-plane and out-of-plane load effects and unbraced walls, which shall be designed either—
 - (i) as a slab in accordance with Section 9. Where the stress at the mid-height section of a wall due to factored in-plane bending and axial forces does not exceed $0.03f'_c$, the wall may be designed as a slab in accordance with Section 9, provided—
 - (A) second-order deflections due to in-plane loads and long-term effects are considered in the calculation of bending moments; and
 - (B) the ratio of effective height to thickness does not exceed 50.

NOTE: The ratio of effective height to thickness may be restricted by fire considerations (see Section 5).
 - (ii) as a column in accordance with Section 10, except that slenderness should be checked using Clause 11.4 to determine k rather than Clause 10.5.3. Clause 11.6 or 8.2 shall be used to check in-plane shear. Out of plane shear shall be checked in accordance with Section 8.

In this section yield strength, f_{sy} shall be not greater than 500 MPa.

11.2 DESIGN PROCEDURES

11.2.1 General

Braced walls where in-plane horizontal forces, acting in conjunction with the axial forces, are such that where a horizontal cross-section of the wall—

- (a) is subject to compression over the entire section, the wall shall be designed for horizontal shear forces in accordance with Clause 11.6 and for the vertical compressive forces either—
 - (i) if within limits defined in Clause 11.5, the wall shall be divided into one or more design segments and each segment shall be designed for the highest stress in that segment in accordance with the simplified method of Clause 11.5; or
 - (ii) as a column in accordance with Section 10 where vertical reinforcement is provided in each face, except that Clause 11.7.4 may override the requirements of Clause 10.7.4. The provisions of Clause 10.7.1(b), Clause 11.4 and Clause 11.7 shall still apply; or
- (b) is subject to tension on part of the section, the wall shall be designed for in-plane bending in accordance with one of the following options as appropriate—
 - (i) if H/L ratio ≤ 2 design as strut and tie according to Section 12. Clause 11.7 still applies; or
 - (ii) if H/L ratio > 2 design as a column in accordance with Section 10 where vertical reinforcement is provided in each face, except that Clause 11.7.4 may override the requirements of Clause 10.7.4. The provisions of Clause 10.7.1(b), Clause 11.4 and Clause 11.7 shall still apply.

A2

The wall shall be designed for horizontal shear forces in accordance with Clause 11.6 or Clause 8.2.

When considering design actions due to earthquake, the determination of whether the horizontal cross-section of a wall is subject to compression over the entire section shall be based on a lateral analysis undertaken using a structural ductility factor (μ) of 1.00 and a structural performance factor (S_p) of 1.0, in accordance with AS 1170.4.

11.2.2 Groups of walls

When two or more walls are directly inter-connected, or linked to each other by coupling beams or the like, the distribution of in-plane load effects between the walls shall be obtained by linear elastic analysis of the overall structure under the global vertical and lateral loads. The analysis shall distribute in-plane load effects to the individual walls in proportion to their relative stiffness, as determined by their gross cross-sectional properties.

The interconnected vertical edges of walls shall be designed to withstand the transmitted vertical shear forces.

11.3 BRACED WALLS

A wall shall be considered to be braced if it forms part of a structure that does not rely on out-of-plane strength and stiffness of the wall and the connection of the wall to the rest of the structure can transmit—

- (a) any calculated load effects; and
- (b) 2.5% of the total vertical load the wall is designed to carry at the level of lateral support, but not less than 2 kN per metre length of wall.

11.4 EFFECTIVE HEIGHT

For a wall with a floor-to-floor unsupported height of H_w and a horizontal length of L_1 between the centres of lateral restraint or from the lateral restraint to the free edge, as applicable, the effective height (H_{we}) shall be taken as $H_{we} = kH_w$, where the factor k is given as follows:

- (a) *One-way buckling* With floors providing lateral support at both ends—

$k = 0.75$ where restraint against rotation is provided at both ends; and

$k = 1.0$ where no restraint against rotation is provided at one or both ends.

- (b) *Two-way buckling* With lateral support on three sides provided by floors and intersecting walls—

$$k = \left[\frac{1}{1 + \left(\frac{H_w}{3L_1} \right)^2} \right] \quad \text{but not less than 0.3 or not greater than what is obtained from Item (a)} \quad \dots 11.4(1)$$

- (c) *Two-way buckling* With lateral support on four sides provided by floors and intersecting walls—

$$k = \frac{1}{1 + (H_w / L_1)^2} \quad \text{where } H_w \leq L_1 \quad \dots 11.4(2)$$

$$k = \frac{L_1}{2H_w} \quad \text{where } H_w > L_1 \quad \dots 11.4(3)$$

Walls supported laterally on four sides that contain one or more openings shall be designed as follows:

- (i) If the total area of the openings is less than 1/10 of the area of the wall and the height of any opening, not vertically one above the other, is less than 1/3 of the height of the wall, then the effect of the openings may be ignored.
- (ii) In other cases—
 - (A) the area of the wall between the support and the opening shall be designed as supported on three sides; and
 - (B) the area between the openings shall be designed as supported on two sides.

NOTE: An intersecting wall with a minimum length of $0.2H_w$ may be considered a lateral restraint.

11.5 SIMPLIFIED DESIGN METHOD FOR WALLS SUBJECT TO VERTICAL COMPRESSION FORCES

11.5.1 General

Where a wall is subject to in-plane bending, the wall shall be divided into one or more design segments and each segment shall be designed for the highest stress in that segment.

11.5.2 Limitation on use of method

Structural walls designed using Clause 11.5.3 shall—

- (a) be limited to a maximum design axial stress of 3 MPa unless vertical and horizontal reinforcement is provided on both wall faces and divided equally between the two wall faces; and
- (b) not be constructed on sites with soil classifications of D_e or E_e , as defined in AS 1170.4, and in a building subject to design earthquake actions; and
- (c) have a ratio of effective height to thickness that does not exceed 20 for singly reinforced walls or 30 for doubly reinforced walls.

Otherwise, the wall shall be designed as a column in accordance with Section 10.

11.5.3 Design axial strength of a wall

Provided the ratio of effective height to thickness (H_{we}/t_w) does not exceed 30, the design axial strength per unit length of a braced wall in compression can be taken as—

$$\phi N_u \quad \dots 11.5.3$$

where

$$\phi = 0.65$$

$$N_u = (t_w - 1.2e - 2e_a) 0.6f'_c$$

N_u = ultimate strength per unit length of wall

t_w = thickness of the wall

e = Eccentricity of the load measured perpendicular to the plane of the wall,
= determined in accordance with Clause 11.5.4.

e_a = an additional eccentricity taken as $(H_{we})^2/2500t_w$

11.5.4 Eccentricity of vertical load

The eccentricity of a vertical load applied to the top of a wall shall be determined as follows:

- (a) The eccentricity of the load applied by the floor (or roof) supported by the top of the wall being designed shall be taken as—
 - (i) for a discontinuous floor, one third of the depth of the bearing area measured from the span face of the wall;
 - (ii) for a cast in situ concrete floor continuous over the wall, zero.
- (b) The eccentricity of the aggregated load from all floors above the floor at the top of the wall being designed may be taken to be zero.

The resultant eccentricity of the total load from (a) and (b) shall be calculated but taken as not less than $0.05t_w$.

11.6 DESIGN OF WALLS FOR IN-PLANE SHEAR FORCES

11.6.1 Critical section for shear

All sections of a wall shall be designed for the shear force at that section. The maximum transverse shear near the base of the wall may be taken as that at a distance $L_w/2$ or $H/2$ from the base, whichever is less.

11.6.2 Strength in shear

The design strength of a wall subject to in-plane shear shall be taken as—

$$\phi V_u \quad \dots 11.6.2$$

where

$$V_u = V_{uc} + V_{us}$$

and V_{uc} and V_{us} are determined from Clauses 11.6.3 and 11.6.4 respectively, but in no case shall V_u be taken as greater than—

$$V_{u,max.} = 0.2f'_c (0.8L_w t_w)$$

The value of ϕ shall be determined from Table 2.2.2.

11.6.3 Shear strength excluding wall reinforcement

The ultimate shear strength of a wall excluding wall reinforcement (V_{uc}) shall be taken as follows:

- (a) For $H/L_w \leq 1$ —

$$V_{uc} = \left(0.66\sqrt{f'_c} - 0.21\frac{H}{L_w}\sqrt{f'_c} \right) 0.8L_w t_w; \text{ or} \quad \dots 11.6.3(1)$$

- (b) For $H/L_w > 1$, the lesser of the values calculated from Item (a) above; and—

$$V_{uc} = \left[0.05\sqrt{f'_c} + \frac{0.1\sqrt{f'_c}}{\left(\frac{H}{L_w} - 1 \right)} \right] 0.8L_w t_w \quad \dots 11.6.3(2)$$

but not less than $0.17\sqrt{f'_c} (0.8L_w t_w)$.

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11.6.4 Contribution to shear strength by wall reinforcement

The contribution to the ultimate shear strength of a wall by wall reinforcement (V_{us}) shall be determined from the following equation:

$$V_{us} = p_w f_{sy} (0.8 L_w t_w) \quad \dots 11.6.4$$

where p_w is determined as follows:

- (a) For walls where $H/L_w \leq 1$, p_w shall be the lesser of the ratios of either the vertical reinforcement area or the horizontal reinforcement area to the cross-sectional area of wall in the respective direction.
- (b) For walls where $H/L_w > 1$, p_w shall be the ratio of the horizontal reinforcement area to the cross-sectional area of wall per vertical metre.

11.7 REINFORCEMENT REQUIREMENTS FOR WALLS

11.7.1 Minimum reinforcement

Walls shall have a reinforcement ratio (p_w)—

- (a) in the vertical direction, of not less than the larger of 0.0025 and the value required for strength unless the design axial compressive force does not exceed the lesser of $0.03 f'_c$ and 2 MPa where the limit may be reduced to 0.0015; and
- (b) in the horizontal direction, of not less than 0.0025, except that for a wall designed for one-way buckling [using Clause 11.4(a)] and where there is no restraint against horizontal shrinkage or thermal movements, this may be reduced to zero if the wall is less than 2.5 m wide, or to 0.0015 otherwise.

NOTE: For walls greater than 500 mm thick, the minimum reinforcement required near each surface may be calculated using 250 mm for t_w .

11.7.2 Horizontal reinforcement for crack control

Where a wall is restrained from expanding or contracting horizontally due to shrinkage or temperature, the horizontal reinforcement ratio shall be not less than the following:

- (a) For exposure classifications A1 and A2—
 - (i) where a minor degree of control over cracking is required 0.0025;
 - (ii) where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view 0.0035; and
 - (iii) where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes 0.006.
- (b) For exposure classifications B1, B2, C1 and C2 0.006.

NOTE: For walls longer than 8 m, additional horizontal crack control reinforcement may be needed at the base of the wall to control restraint cracking during hydration and shrinkage.

11.7.3 Spacing of reinforcement

The minimum clear distance between parallel bars, ducts and tendons shall be sufficient to ensure the concrete can be placed and compacted to conform with Clause 17.1.3 but shall be not less than $3d_b$.

The maximum centre-to-centre spacing of parallel bars shall be the lesser of $2.5t_w$ and 350 mm.

The vertical and horizontal reinforcement shall be provided in two grids, one near each face of the wall under any of the following conditions:

- (a) Walls greater than 200 mm thick.
- (b) Any part of a wall structure where tension exceeds the tensile capacity of the concrete under the design ultimate loads. When considering design actions due to earthquake, the determination of whether or not a wall horizontal cross-section is subject to tension shall be based on a lateral analysis undertaken using a structural ductility factor (μ) of 1.00 and a structural performance factor (S_p) of 0.77.
- (c) Walls designed for two-way buckling [based on Clauses 11.4(b) or 11.4(c)].
- (d) Walls exceeding 20 metres in total height or with slenderness ratios of greater than 20.

11.7.4 Restraint of vertical reinforcement

In addition to providing transverse reinforcement required for any design actions, the following restraint to vertical reinforcement provisions shall be satisfied:

- (a) For all walls in structures with a structural ductility factor (μ) greater than 1.0, the vertical reinforcement shall be restrained in accordance with Clause 14.6.
- (b) For walls with concrete strength not exceeding 50 MPa and designed as columns in accordance with Section 10, the vertical reinforcement shall be restrained in accordance with Clause 10.7.4 unless one or more of the following conditions is met, in which case no restraint is required:
 - (i) $N^* \leq 0.5 \phi N_u$.
 - (ii) The vertical reinforcement is not used as compressive reinforcement.
 - (iii) The vertical reinforcement ratio is not greater than 0.01 and a minimum horizontal reinforcement ratio of 0.0025 is provided.
- (c) For walls with concrete strength not exceeding 50 MPa and designed using the simplified method in accordance with Clause 11.5, no restraint of vertical reinforcement is required.
- (d) For walls with a concrete strength exceeding 50 MPa, the core of the wall shall be confined, and the vertical reinforcement shall be restrained as follows:
 - (i) In accordance with Clause 14.5.4 for walls designed as columns in accordance with Section 10 where $N^* > 0.15 f'_c A_g$ and $M^* > 0.6 \phi M_u$.
 - (ii) In accordance with Clause 14.5.4 for walls designed using the simplified method in accordance with Clause 11.5 where N^* in any portion of the wall is greater than $0.5 \phi N_u$.
 - (iii) For all other scenarios:
 - (A) Provide U bars, fully lapped with the horizontal reinforcement at the ends of the walls and corners of the walls.
 - (B) U bars shall have a maximum spacing of the lesser of the wall thickness t_w and 200 mm.
 - (C) Regions over a distance of the greater of $2t_w$ and $0.15L_w$ from the ends of the wall and corners required to have ties as specified in Clauses 10.7.3 and 10.7.4.

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11.7.5 Dowelled connections in prefabricated concrete walls

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Dowels shall be provided to satisfy requirements for axial force, bending, horizontal shear and any other load effects at the joint. Dowels shall meet the following minimum requirements:

- (a) Where structural ductility factor (μ) greater than 1.0 has been used in the earthquake analysis, dowels shall be capable of transferring the yield force of the vertical wall reinforcement in the wall through the joint. As a minimum, $A_{st,dowel}$ shall be greater than $A_{st,wall}$.
- (b) For earthquake design actions using a structural ductility factor (μ) of 1.0 and a structural performance factor (S_p) of 0.77 or greater, and for any other non-earthquake loading scenarios (e.g. wind actions):
 - (i) For walls that remain in net compression there are no special requirements.
 - (ii) For walls that do not remain in net compression, the area of dowel reinforcement shall be equal to at least 50% of the wall reinforcement, $A_{st,dowel} \geq 0.5A_{st,wall}$.

The dowel reinforcement shall always be greater than the amount corresponding to the minimum reinforcement ratio calculated using Clause 11.7.1 and all dowels shall be fully anchored in accordance with Section 13, regardless of the above scenarios.

SECTION 12 DESIGN OF NON-FLEXURAL MEMBERS, END ZONES AND BEARING SURFACES

12.1 GENERAL

12.1.1 Scope of Section

This section applies to the design of non-flexural members, including deep beams, footings, and pile caps where the ratio of the clear span or projection to the overall depth is less than—

- (a) for cantilevers.....1.5;
- (b) for simply supported members 3; and
- (c) for continuous members.....4.

This section also applies to the design of non-flexural regions, including corbels, continuous nibs, end zones of prestressed members, and surfaces where concentrated forces act.

12.1.2 Design for strength

The design for strength shall be carried out using one of the following:

- (a) Linear elastic stress analysis and the checking procedure given in Clause 2.2.3.
- (b) Strut-and-tie analysis, and the checking procedure given in Clause 2.2.4.
- (c) Non-linear stress analysis and the checking procedure given in Clause 2.2.6.

The value of the capacity reduction factor shall be determined from Clause 2.2, for the analysis and checking procedure adopted.

12.1.3 Design for serviceability

Design for serviceability shall be in accordance with Clauses 2.3 and 12.7.

12.2 STRUT-AND-TIE MODELS FOR THE DESIGN OF NON-FLEXURAL MEMBERS

12.2.1 Design models

Design models are distinguished by the method in which the forces are transferred from the point of loading to the supports. The models are identified as Types I, II and III. These are shown in Figure 12.2.1 for the specific case of deep beams, and are defined as follows:

- (a) *Type I* The load is carried to the supports directly by major struts.
- (b) *Type II* The load is taken to the supports by a combination of primary (major) and secondary (minor) struts. Adequately anchored hanger reinforcement is required to return the vertical components of forces developed in the secondary struts to the top of the member.
- (c) *Type III* The load is carried to the supports via a series of minor struts with adequately anchored hanger reinforcement used to return the vertical components of the strut forces to the top of the member.

Hanger reinforcement in the form of fitments shall be anchored in accordance with Clause 8.3.2.4.

For Type II models, the force carried by the secondary struts shall be within the limits $0 \leq T_w \leq F$, where T_w is the vertical component of the force carried by the secondary struts and F is the total vertical component of the external load carried through the shear span.

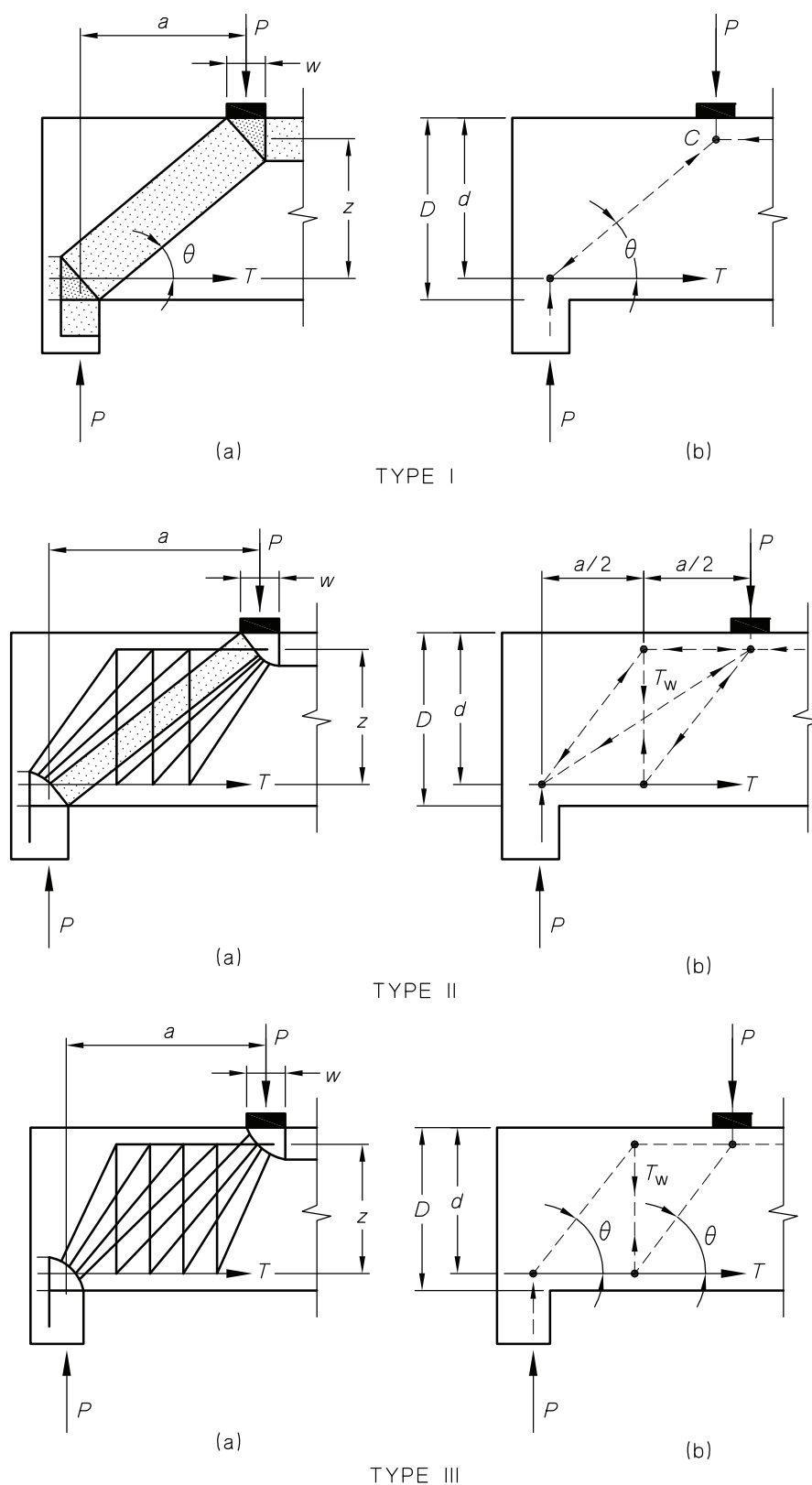


FIGURE 12.2.1 (a) STRUT-AND-TIE MODELS AND (b) SIMPLIFIED DESIGN MODELS

12.2.2 Strut bursting reinforcement

Strut bursting reinforcement shall be provided in accordance with Clause 7.2.4.

12.3 ADDITIONAL REQUIREMENTS FOR CONTINUOUS CONCRETE NIBS AND CORBELS

Corbels and continuous nibs that support other members shall be also designed to conform with the following:

- (a) The tensile reinforcement shall be anchored at the free end of the nib or corbel, either by a welded or mechanical anchorage, or by a loop in either the vertical or horizontal plane. Where the main reinforcement is looped, the loaded area shall not project beyond the straight portion of this reinforcement.
- (b) Horizontal forces resulting from the supported member, because of factors, such as movement, shrinkage, temperature and prestress, shall be assessed but shall not be taken as less than 20% of the vertical force.
- (c) The line of action of the load shall be taken at the outside edge of a bearing pad for continuous nibs and at one third the width of the bearing from the free end for a corbel. Where no bearing pad is provided, the line of action may be taken at the commencement of any edge chamfer, or at the outside face of the nib or corbel.
- (d) Where a flexural member is being supported, the outside face of a nib shall be protected against spalling.

12.4 ADDITIONAL REQUIREMENTS FOR STEPPED JOINTS IN BEAMS AND SLABS

The design of stepped joints shall take into account the horizontal forces and movements from the supported members and shall conform with the following:

- (a) Horizontal forces resulting from movement, shrinkage, temperature, prestress and other factors in the supported member shall be assessed but shall not be taken as less than 20% of the vertical force.
- (b) In prestressed members, the vertical component of the force from the prestressing steel shall be ignored.
- (c) The horizontal reinforcement shall extend at least a distance equal to the beam depth (D) beyond the step and shall be provided with anchorage beyond the plane of any potential shear crack.
- (d) Hanging reinforcement shall be placed as close as possible to the vertical face of the step.

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12.5 BEARING ZONES BEHIND CONCENTRATED FORCES AND PRESTRESSING ANCHORAGES

12.5.1 General

Components subjected to concentrated forces, including anchorage zones in post-tensioned concrete members, shall be designed in accordance with Section 7. Alternatively, this section may be used for post-tensioned anchorage zones in cases having no more than two anchorages in any elevation or plan.

NOTE: Where there are more than two anchorages in any elevation or plan, the design may be undertaken in accordance with Section 7.

A2 12.5.2 Reinforcement

Reinforcement shall be provided to carry tensile forces that arise from the action and dispersal of concentrated loads or prestressing forces in the zone behind the point of bearing.

In general, the dispersal occurs through both the depth and the width of the anchorage or bearing zone, and reinforcement shall, therefore, be provided in planes parallel to the end faces in two orthogonal directions. A two-dimensional analysis for each loading case shall be carried out in each direction in turn. The tensile forces shall be calculated on longitudinal sections through anchorages or bearing zones and on longitudinal sections where peak values of transverse moments occur.

The transverse moment on a longitudinal section is the equilibrating moment acting on the free body bounded by the longitudinal section, a free surface parallel to it, the loaded face, and a plane parallel to the loaded face at the inner end of the anchorage or bearing zone.

12.5.3 Loading cases to be considered for prestressing systems

Loading cases to be considered shall include—

- (a) all anchorages loaded; and
- (b) critical loadings during the stressing operation.

Where the distance between two anchorages is less than 0.3 times the total depth, or breadth, of the member, consideration shall be given to the effects of the pair acting in a manner similar to a single anchorage subject to the combined forces.

12.5.4 Calculation of tensile forces along the line of a concentrated load or anchorage force

A2 The force resultant of transverse tensile stresses induced along the line of action of an anchorage force or concentrated load shall be taken as follows:

$$T = 0.25P (1 - k_r) \quad \dots 12.5.4$$

where

P = the maximum applied load (for prestressing systems equals the maximum load applied during jacking)

k_r = ratio of the depth, or breadth, of a bearing plate to the corresponding depth, or breadth, of the symmetrical prism

The symmetrical prism is defined as a notional prism with an anchorage at the centre of its end face and a depth, or breadth, taken as twice the distance from the centre of an anchorage or bearing plate to the nearer concrete face.

12.5.5 Calculation of tensile forces induced near the loaded face

A2 At longitudinal sections remote from a single eccentric anchorage or bearing plate, or between widely spaced anchorages, where the sense of the transverse moment indicates the tensile stress resultant acts near the loaded face, the tensile force shall be calculated as follows:

- (a) For a single eccentric anchorage or bearing plate, by dividing the peak transverse moment by a lever arm assumed to be one half the overall depth of the member.
- (b) Between pairs of anchorages, by dividing the peak transverse moment by a lever arm assumed to be 0.6 times the spacing of the anchorages.

12.5.6 Quantity and distribution of reinforcement

The cross-sectional area of reinforcement for each situation shall be calculated by dividing the tensile forces derived in accordance with Clauses 12.5.4 and 12.5.5 by 150 MPa. This reinforcement shall be distributed as follows:

- (a) Reinforcement to resist the forces calculated under Clause 12.5.4 distributed uniformly from $0.2D$ to $1.0D$ from the loaded face. Similar reinforcement shall be placed from the plane at $0.2D$ to as near as practicable to the loaded face. D shall be equal to the depth or breadth of the symmetrical prism.
- (b) Having side slopes of one longitudinally to one transversely with respect to the direction of the load.

At any plane parallel to the loaded face, the reinforcement shall be determined from the longitudinal section with the greatest reinforcement requirements at that plane, and shall extend over the full depth or breadth of the end zone.

12.6 BEARING SURFACES

Unless special confinement reinforcement is provided, the maximum design bearing stress at a concrete surface shall not exceed—

$$\phi 0.9 f'_c \sqrt{(A_2 / A_1)} \text{ or } \phi 1.8 f'_c \quad \dots 12.6$$

whichever is less—

where

A_2 = largest area of the supporting surface that is geometrically similar to and concentric with A_1

A_1 = a bearing area

In the case of a bearing surface where the supporting structure is sloped or stepped, it shall be permissible to take A_2 as the area of the base of the largest frustum of a right pyramid or cone—

- (a) having for its opposite end the bearing area A_1 ;
- (b) having side slopes of one longitudinally to two transversely, with respect to the direction of the load; and
- (c) contained wholly within the supporting structure.

This clause is not applicable to the design of nodes within a strut-and-tie model.

12.7 CRACK CONTROL

The requirements for crack control shall be deemed to be satisfied if the stress in the reinforcement under the short-term service loads σ_{st} and the maximum centre-to-centre spacing between the bonded reinforcement crossing the crack s satisfy the following:

- (a) Where a minor degree of control over cracking is required $\sigma_{st} \leq 350$ MPa.
 $s \leq 350$ mm.
- (b) Where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view $\sigma_{st} \leq 250$ MPa.
 $s \leq 300$ mm.
- (c) Where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes $\sigma_{st} \leq 200$ MPa.
 $s \leq 200$ mm.

For prestressed concrete, the change in stress in the tendons after the point of decompression shall not exceed the limits given by Items (a), (b) or (c).

SECTION 13 STRESS DEVELOPMENT OF REINFORCEMENT AND TENDONS

13.1 STRESS DEVELOPMENT IN REINFORCEMENT

13.1.1 General

The calculated force in reinforcing steel at any cross-section shall be developed on each side of that cross-section in accordance with Clauses 13.1.2 to 13.1.8.

13.1.2 Development length for a deformed bar in tension

13.1.2.1 Development length to develop yield strength

The development length ($L_{sy,t}$) to develop the characteristic yield strength (f_{sy}) of a deformed bar in tension shall be calculated from either Clause 13.1.2.2 or Clause 13.1.2.3.

13.1.2.2 Basic development length

The development length ($L_{sy,t}$) shall be taken as the basic development length of a deformed bar in tension ($L_{sy,tb}$), calculated from—

$$L_{sy,tb} = \frac{0.5k_1k_3f_{sy}d_b}{k_2\sqrt{f'_c}} \geq 0.058f_{sy}k_1d_b \quad \dots 13.1.2.2$$

where

- $k_1 = 1.3$ for a horizontal bar with more than 300 mm of concrete cast below the bar; or
- $= 1.0$ otherwise
- $k_2 = (132 - d_b)/100$, and
- $k_3 = 1.0 - 0.15(c_d - d_b) / d_b$ (within the limits $0.7 \leq k_3 \leq 1.0$); where
 c_d = a dimension (in millimetres), as shown in Figure 13.1.2.2

The value of f'_c used in Equation 13.1.2.2 shall not be taken to exceed 65 MPa; and the bar diameter (d_b) is in millimetres.

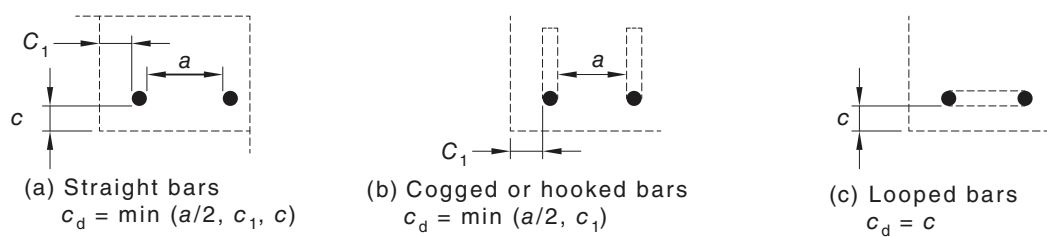
The value of $L_{sy,tb}$ calculated as above shall be—

- (a) multiplied by 1.5 for epoxy-coated bars; and
- (b) multiplied by 1.3 when lightweight concrete is used.

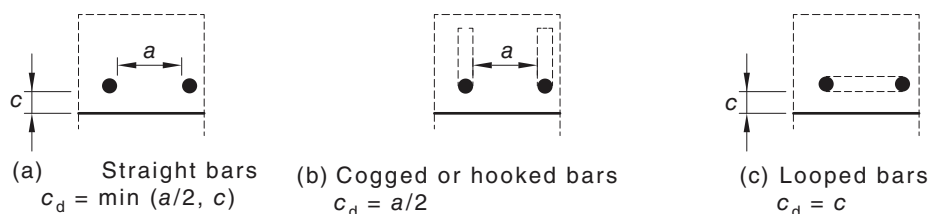
NOTE: A smaller value of $L_{sy,t}$ may be possible using the provisions of Clause 13.1.2.3.

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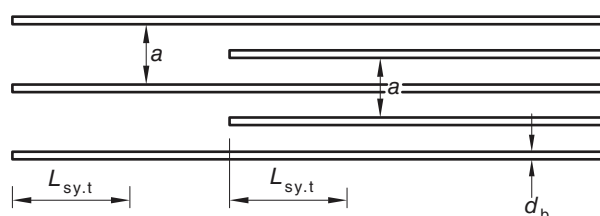
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(i) Narrow elements or members (e.g. beam webs and columns)



(ii) Wide elements or members (e.g. flanges, band beams, slabs, walls and blade columns)



(iii) Planar view of staggered development lengths of equi-spaced bars

FIGURE 13.1.2.2 VALUES OF c_d

13.1.2.3 Refined development length

Where a refined development length is required, the development length in tension ($L_{sy,t}$) shall be calculated from—

$$L_{sy,t} = k_4 k_5 L_{sy,tb} \quad \dots 13.1.2.3$$

where

$$k_4 = 1.0 - K\lambda \quad (\text{within the limits } 0.7 \leq k_4 \leq 1.0)$$

$$k_5 = 1.0 - 0.04\rho_p \quad (\text{within the limits } 0.7 \leq k_5 \leq 1.0)$$

K = a factor that accounts for the weighted average effectiveness of transverse reinforcement in controlling potential splitting cracks along a development or lap splice length (typical values of K are given in Figure 13.1.2.3);

$$= 0.05 \times (1 + n_f/n_{bs}) \leq 0.10$$

with values of n_f and n_{bs} shown in Table 13.1.2.3 for typical arrangements of transverse reinforcement for different member types; and

= 0 if the transverse reinforcement is not located between the longitudinal bars and the concrete tensile face

$$\lambda = (\Sigma A_{tr} - \Sigma A_{tr,min})/A_s \geq 0$$

ΣA_{tr} = sum of cross-sectional area of the transverse bars along a development or lap length

$\Sigma A_{tr,min}$ = sum of cross-sectional area of the minimum transverse reinforcement, which may be taken as $0.25A_s$ for beams or columns and 0 for slabs or walls

A_s = cross-sectional area of a single bar of diameter (d_b) being anchored

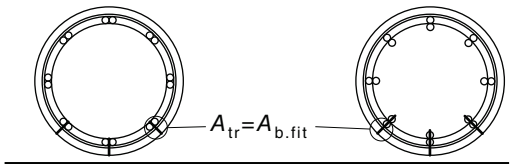
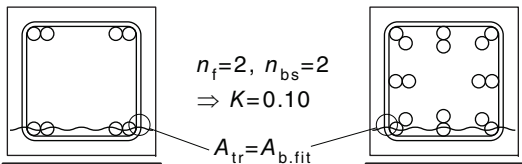
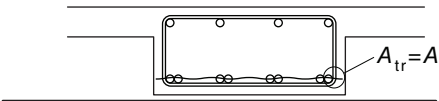
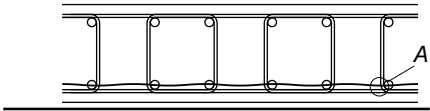
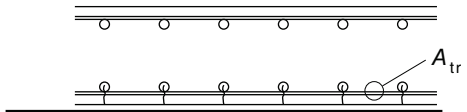
ρ_p = transverse compressive pressure (in MPa), at the ultimate limit state along the development length and perpendicular to the plane of splitting

The product $k_3 k_4 k_5$ shall be not taken as less than 0.7.

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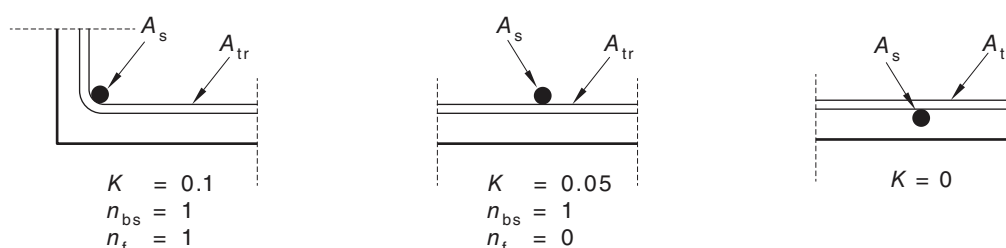
TABLE 13.1.2.3

VALUES OF K FOR TYPICAL ARRANGEMENTS OF TRANSVERSE REINFORCEMENT FOR DIFFERENT MEMBER TYPES

Member type	Examples of potential splitting cracks at a tensile face	n_f	n_{bs}	K (see Note 2)
Circular column	 $A_{tr} = A_{b.fit}$	1	1	0.10
Rectangular column	 $n_f = 2, n_{bs} = 2 \Rightarrow K = 0.10$ $n_f = 2, n_{bs} = 3 \Rightarrow K = 0.083$ $A_{tr} = A_{b.fit}$	≥ 1	≥ 1	$0.05 \leq K \leq 0.10$
Beam	 $n_f = 2, n_{bs} = 4 \Rightarrow K = 0.075$ $A_{tr} = A_{b.fit}$	≥ 1	≥ 1	$0.05 \leq K \leq 0.10$
Slab or wall (with fitments)	 $n_f = n_{bs} \Rightarrow K = 0.10$ $A_{tr} = A_{b.fit}$	≥ 1	≥ 1	$0.05 \leq K \leq 0.10$
Slab or wall (without fitments) ¹	 A_{tr}	0	1 per main bar spacing	0.05 (see Note 3)

NOTES:

- Fitments are a type of transverse reinforcement.
- The same value of K applies to all of the longitudinal bars being either anchored or lap spliced, that is, it is a weighted average value.
- To be effective, the transverse reinforcement needs to be located between the longitudinal bars and the concrete tensile face as shown, otherwise $K = 0$.

FIGURE 13.1.2.3 VALUES OF K FOR DIFFERENT BAR POSITIONS**13.1.2.4** *Development length to develop less than the yield strength*

Where the full yield strength of the bar is not required, the development length (L_{st}) to develop a tensile stress (σ_{st}), less than the yield strength (f_{sy}), shall be calculated from—

$$L_{st} = L_{sy,t} \frac{\sigma_{st}}{f_{sy}} \quad \dots 13.1.2.4$$

but shall be not less than—

- (a) $12d_b$; or
- (b) for slabs, as permitted by Clause 9.1.3.1(a)(ii).

NOTE: When calculating σ_{st} account should be taken of possible redistribution of actions in regions near potential hinge locations in the structure.

13.1.2.5 *Development length around a curve*

Tensile stress may be considered to be developed around a curve if the internal diameter of the curve is $10d_b$ or greater.

13.1.2.6 *Development length of a deformed bar with a standard hook or cog*

Where a deformed bar ends in a standard hook or cog conforming with Clause 13.1.2.7, the tensile development length of that end of the bar, measured from the outside of the hook/cog, shall be taken as $0.5L_{sy,t}$ (as shown in Figure 13.1.2.6), with L_{st} determined from using Equation 13.1.2.4 and the maximum stress in the bar being anchored σ_{st} . Where σ_{st} exceeds 400 MPa, the hook or cog must enclose a transverse bar with a diameter d_b not less than the diameter of the bar being anchored. The enclosed transverse bar shall be in contact with the hook/cog and extend at least $4d_b$ on either side of the hook/cog.

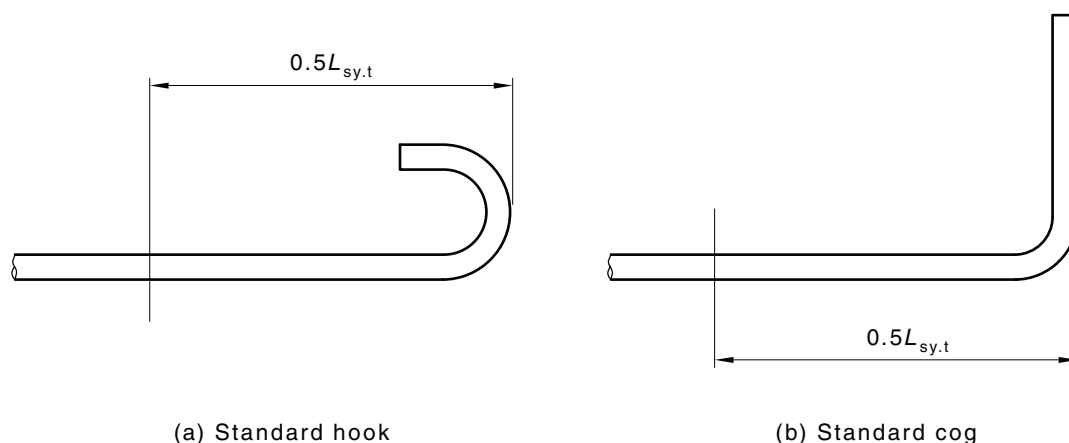


FIGURE 13.1.2.6 DEVELOPMENT LENGTH OF A DEFORMED BAR WITH A STANDARD HOOK OR COG

13.1.2.7 *Standard hooks and cogs*

The standard hook or cog referred to in Clause 13.1.2.6 shall be one of the following:

- A2 | (a) A hook consisting of a 180° bend with a nominal internal diameter conforming with Clause 17.2.3.3 plus a straight extension of $4d_b$ or 70 mm, whichever is greater.
- (b) A hook consisting of a 135° bend with the same internal diameter and length as Item (a).
- A2 | (c) A cog, consisting of a 90° bend with a nominal internal diameter conforming with Clause 17.2.3.3 but not greater than $8d_b$ and having the same total length as required for a 180° hook of the same diameter bar.

13.1.3 Development length of plain bars in tension

The development length ($L_{sy,t}$) to develop the yield strength (f_{sy}) of a plain bar in tension shall be taken as the basic development length calculated in accordance with Clause 13.1.2.2 multiplied by 1.5, but $L_{sy,t}$ shall be not less than 300 mm.

Where a plain bar ends in a standard hook or cog conforming with Clause 13.1.2.7, the tensile development length of that end of the bar, measured from the outside of the hook/cog, shall be taken as $0.5L_{sy,t}$ or $0.5L_{st}$ as applicable (as shown in Figure 13.1.2.6).

13.1.4 Development length of headed reinforcement in tension

A head used to develop a deformed bar in tension shall be either—

- (a) directly forged onto the bar; or
- (b) shall consist of a nut or plate, having either a round, elliptical or rectangular shape, attached to the end(s) of the bar by welding, threading or swaging of suitable strength to avoid failure of the steel connection at ultimate load.

In addition—

- (i) the net bearing area of the head (A_{head}) shall be not less than four times the cross-sectional area of the bar A_{bar} ;
- (ii) the bar diameter (d_b) shall not exceed 40 mm;
- (iii) the clear cover for the bar shall not be less than $2d_b$; and
- (iv) the clear spacing between bars shall be not less than $4d_b$.

The headed bar shall be considered to have a development length ($L_{sy,hb}$) measured from the inside face of the head equal to—

- (A) when $A_{head}/A_{bar} = 4$ $L_{sy,hb} = 0.5 L_{sy,t}$
 (B) when $A_{head}/A_{bar} \geq 10$ $L_{sy,hb} = 6 d_b$

For values of A_{head}/A_{bar} between 4 and 10, the headed bar $L_{sy,hb}$ shall be obtained by linear interpolation.

Where the tensile force in a headed bar could give rise to bearing forces directed towards, or adjacent to, a free concrete surface, failure of the concrete cone between the head of the bar and the free concrete surface shall be investigated.

13.1.5 Development length of deformed bars in compression

13.1.5.1 Development length to develop yield strength

The development length ($L_{sy,c}$) to develop the characteristic yield strength (f_{sy}) of a deformed bar in compression shall be calculated from either Clause 13.1.5.2 or Clause 13.1.5.3, but shall be not less than 200 mm.

13.1.5.2 Basic development length

The development length ($L_{sy,c}$) shall be taken as the basic development length of a deformed bar in compression ($L_{sy,cb}$) calculated from—

$$L_{sy,cb} = \frac{0.22 f_{sy}}{\sqrt{f'_c}} d_b \geq 0.0435 f_{sy} d_b \text{ or } 200 \text{ mm, whichever is the greater} \quad \dots 13.1.5.2$$

NOTE: A smaller value of $L_{sy,c}$ may be obtained using the provisions of Clause 13.1.5.3.

13.1.5.3 Refined development length

Where a refined development length is required, the development length in compression ($L_{sy,c}$) shall be calculated from—

$$L_{sy,c} = k_6 L_{sy,cb} \quad \dots 13.1.5.3$$

Where transverse reinforcement with at least three bars, transverse to and outside the bar being developed is provided within $L_{sy,cb}$ and when $\Sigma A_{tr}/s \geq A_s/600$,

$$k_6 = 0.75$$

where ΣA_{tr} and A_s are defined in Clause 13.1.2.3.

In all other cases, $k_6 = 1.0$.

13.1.5.4 Development length to develop less than the yield strength

The development length (L_{sc}) to develop a compressive stress (σ_{sc}), less than the yield strength (f_{sy}), shall be calculated from—

$$L_{sc} = L_{sy,c} \frac{\sigma_{sc}}{f_{sy}} \text{ (but not less than 200 mm)} \quad \dots 13.1.5.4$$

A bend or a standard hook shall not be considered effective in developing stress in reinforcement in compression.

13.1.6 Development length of plain bars in compression

The development length for plain bars in compression shall be twice the calculated value of $L_{sy,c}$ or $L_{sy,cb}$ for a deformed bar.

13.1.7 Development length of bundled bars

The development length of a unit of bundled bars shall be based on the development length required for the largest bar within the bundle increased by—

- (a) for a 3-bar bundle 20%; and
- (b) for a 4-bar bundle 33%.

13.1.8 Development length of welded plain or deformed mesh in tension

13.1.8.1 Development length to develop yield strength

The development length ($L_{sy.tb}$) of welded plain or deformed mesh, measured from the critical section to the end of the bar or wire, shall be calculated in accordance with Clause 13.1.8.2, Clause 13.1.8.3 or Clause 13.1.8.4.

13.1.8.2 Two or more cross-bars within development length

The yield strength of plain or deformed bars of welded mesh shall be considered to be developed by embedding at least two cross-bars spaced at not less than 100 mm or 50 mm apart within the development length for plain or deformed bars respectively, with the first one not less than 50 mm from the critical section.

13.1.8.3 One cross-bar within development length

When only one cross-bar is located within the development length, the minimum length measured from the critical section to the outermost cross-bar shall be not less than $L_{sy.tb}$ calculated from—

$$L_{sy.tb} = 3.25 \frac{A_b}{s_m} \frac{f_{sy}}{\sqrt{f'_c}} \quad \dots 13.1.8.3$$

but not less than 150 mm for plain mesh and not less than 100 mm for deformed mesh, where

A_b = area of the individual bar being developed in square millimetres

s_m = spacing of bars being developed, in millimetres

13.1.8.4 No cross-bars within development length

When no cross-bars are located within the development length, the development length of welded mesh shall be determined by Clauses 13.1.2 and 13.1.3.

13.1.8.5 Development length to develop less than the yield strength

The development length (L_{st}) to develop a tensile stress (σ_{st}) less than the yield strength (f_{sy}) shall be calculated from the development length of Clauses 13.1.8.3 or 13.1.8.4 using the following equation:

$$L_{st} = L_{sy.tb} \frac{\sigma_{st}}{f_{sy}} \quad \dots 13.1.8.5$$

but not less than 150 mm for plain mesh and not less than 100 mm for deformed mesh.

13.2 SPLICING OF REINFORCEMENT

13.2.1 General

The following general requirements shall apply to the splicing of reinforcement:

- (a) Splices of reinforcement shall be made only as required or permitted on the design drawings or in specifications.
- (b) The splice shall be made by welding, by mechanical means, by end-bearing, or by lapping.
- (c) Splicing of reinforcement shall take into account the requirements of Clause 17.1.3 regarding the placement of concrete.

- (d) Splices required in bars in tension-tie members shall be made only by welding or mechanical means.
- (e) Lapped splices shall not be used for bars in compression or tension with diameter larger than 40 mm.
- (f) Welding of reinforcing bars shall not be made less than $3d_b$ from that part of a bar that has been bent and re-straightened.

13.2.2 Lapped splices for bars in tension

In wide elements or members (e.g. flanges, band beams, slabs, walls and blade columns), where the bars being lapped are in the plane of the element or member, the tensile lap length ($L_{sy.t.lap}$) for either contact or non-contact splices shall be calculated from—

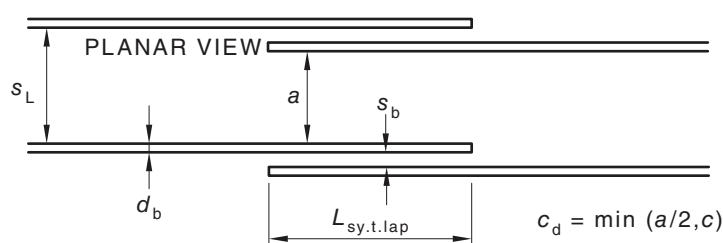
$$L_{sy.t.lap} = k_7 L_{sy.t} \geq 0.058 f_{sy} k_1 d_b \quad \dots 13.2.2$$

where

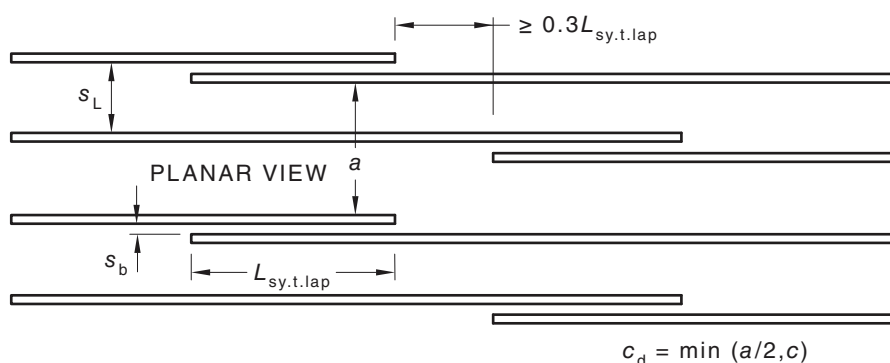
$L_{sy.t}$ is calculated in accordance with Clause 13.1.2.1. (In the determination of $L_{sy.t}$ for use in Equation 13.2.2, the lower limit of $0.058 f_{sy} k_1$ in Equation 13.1.2.2 does not apply); and

k_7 shall be taken as 1.25 unless A_s provided is at least twice A_s required and no more than half of the reinforcement at the section is spliced, in which case k_7 may be taken as 1.

In narrow elements or members (such as beam webs and columns), the tensile lap length ($L_{sy.t.lap}$) shall be not less than the larger of $0.058 f_{sy} k_1$, $k_7 L_{sy.t}$ and $L_{sy.t} + 1.5s_b$, where s_b is the clear distance between bars of the lapped splice as shown in Figure 13.2.2. However, if s_b does not exceed $3d_b$, then s_b may be taken as zero for calculating $L_{sy.t.lap}$.



(i) 100% of bars spliced (no staggered splice)



(ii) 50% staggered splices

FIGURE 13.2.2 VALUE OF c_d FOR LAPPED SPLICES

13.2.3 Lapped splices for mesh in tension

A lapped splice for welded mesh in tension shall be made so the two outermost cross-bars spaced at not less than 100 mm or 50 mm apart for plain or deformed bars, respectively, of one sheet of mesh overlap the two outermost cross-bars of the sheet being lapped as shown in Figure 13.2.3. The minimum length of the overlap shall equal 100 mm.

A lapped splice for welded deformed and plain meshes, with no cross-bars within the splice length shall be determined in accordance with Clause 13.2.2.



FIGURE 13.2.3 LAPPED SPLICES FOR WELDED MESH

13.2.4 Lapped splices for bars in compression

The minimum length of a lapped splice for deformed bars in compression shall be the development length in compression ($L_{sy,c}$) given in Items (a), (b) or (c), but shall be not less than 300 mm:

- (a) The development length in compression shall be in accordance with Clause 13.1.5 but not less than $40d_b$.
- (b) In compressive members with stirrups or fitments where at least three sets of fitments are present over the length of the lap and $A_{tr}/s \geq A_b/1000$, a lap length of 0.8 times the value given in Item (a).
- (c) In helically tied compressive members, if at least three turns of helical reinforcement are present over the length of the lap and $A_{tr}/s \geq n A_b/6000$, a lap length of 0.8 times the value given in Item (a), where n = the number of bars uniformly spaced around the helix.

In this clause, A_b is defined as the area of the bar being spliced.

13.2.5 Lapped splices for bundled bars

Lapped splices for a unit of bundled bars shall be based on the lap splice length required for the largest bar within the bundle increased by—

- (a) for a three bar bundle..... 20%; and
- (b) for a four bar bundle 33%.

Individual bar splices within a bundle shall not overlap.

13.2.6 Welded or mechanical splices

Welded or mechanical splices formed between Ductility Class N reinforcing bars shall not fail prematurely in tension or compression before the reinforcing bars, unless it can be shown that the strength and ductility of the concrete member meets the design requirements.

When control of cracking or vertical deflection are relevant serviceability design criteria, the potentially detrimental effects of excessive longitudinal slip between spliced Ductility Class N bars and a proprietary mechanical connector shall be considered if tests show the effective slip in the assemblage could exceed 0.1 mm at a tensile stress of 300 MPa. The effective slip shall be taken as the overall deformation of a spliced pair of reinforcing bars measured over a gauge length of $12d_b$, less the elongation of the bars assuming they are unspliced over the same gauge length.

13.3 STRESS DEVELOPMENT IN TENDONS

13.3.1 General

In the absence of substantiated test data, the length to develop the calculated force in a pretensioned tendon shall be taken to be a bi-linear relationship defined by the transmission length (L_{pt}) in Clause 13.3.2.1 and the total development length (L_p) in Clause 13.3.2.2.

13.3.2 Transmission lengths of pretensioned tendons

13.3.2.1 Transmission lengths of pretensioned tendons

The transmission length required to develop the effective prestress in pretensioned tendons shall be taken as the length given in Table 13.3.2, appropriate to type of tendon and strength of concrete at transfer. The transmission length shall be taken to be independent of the effective prestress in the tendon.

It shall be assumed that no change in the position of the inner end of the transmission length occurs with time but that a completely unstressed zone of length $0.1L_{pt}$ develops at the end of the tendon.

TABLE 13.3.2
MINIMUM TRANSMISSION LENGTH
FOR PRETENSIONED TENDONS

Type of tendon	L_{pt} for gradual release	
	$f_{cp} \geq 32 \text{ MPa}$	$f_{cp} < 32 \text{ MPa}$
Indented wire	$100 d_b$	$175 d_b$
Crimped wire	$70 d_b$	$100 d_b$
Ordinary and compact strand	$60 d_b$	$60 d_b$

13.3.2.2 Development length of pretensioned strand

In absence of test data, the bonded length to develop the stress in seven wire pretensioned strand at ultimate strength shall be taken as not less than—

$$L_p = 0.145(\sigma_{pu} - 0.67\sigma_{p,ef})d_b \geq 60d_b \quad \dots 13.3.2.2$$

where $\sigma_{p,ef}$ is the effective stress in the tendon after allowing for all losses.

Both σ_{pu} and $\sigma_{p,ef}$ are in megapascals, and the expression in parenthesis is used without units.

Embedment less than the development length is permitted at a section of a member provided the design stress in the strand at that section does not exceed the values obtained from the bi-linear relationship defined by this clause and Clause 13.3.2.1.

The development length of de-bonded strand shall be taken to be $2L_p$ where the design includes tension in accordance with Clauses 8.6.2 and 9.4.2 in the development length.

13.3.2.3 *Development length of pretensioned wire*

Pretensioned indented and crimped wire tendons shall be bonded beyond the critical section for a length sufficient to develop the design stress in the wire but not less than 2.25 times the value for the transmission length in Table 13.3.2.

13.3.2.4 *Development length of untensioned strand or wire*

A2 | Where strand or wire is untensioned, the development length shall be taken as not less than 2.5 times the value of the transmission length of a stressed tendon given in Table 13.3.2 for a tendon stressed to the characteristic minimum breaking strength (f_{pb}) in Table 3.3.1.

13.3.3 **Stress development in post-tensioned tendons by anchorages**

A2 | Anchorages for tendons shall be capable of developing in the tendon the characteristic minimum breaking strength (f_{pb}).

In addition, anchorages for unbonded tendons shall be capable of sustaining cyclic loading conditions.

13.4 **COUPLING OF TENDONS**

Coupling of tendons shall conform with the following:

- (a) Couplers shall be capable of developing at least 95% of the tendon characteristic minimum breaking force specified.
- (b) Couplers shall be enclosed in grout-tight housings to facilitate grouting of the duct.

SECTION 14 DESIGN FOR EARTHQUAKE ACTIONS

14.1 GENERAL

This section applies to concrete structures and structural members that contain reinforcement, or tendons, or both, and which form the whole or part of structures or buildings to which AS 1170.4 applies.

Plain concrete members and structures shall not be used to resist earthquake actions, except plain concrete pedestals, footings and pavements are deemed to satisfy the requirements of Clause 2.1.2 and this section.

Values for the structural ductility factor (μ) and the structural performance factor (S_p) for concrete members and structures conforming with this Standard shall be as given in Clause 14.3.

Concrete structures and members shall be designed and detailed depending on the value adopted for the structural ductility factor (μ) as follows:

All members shall be designed and detailed—

- (a) in accordance with Clause 14.4; and
- (b) the required option below depending on the value of μ —
 - (i) for $1 < \mu \leq 3$ designed and detailed in accordance with this Standard (AS 3600) and this section, as appropriate;
 - (ii) for $\mu > 3$ the structure should be designed and detailed in accordance with NZS 1170.5, NZS 3101 and the AS 1170.4 Hazard Map. This will require the use of Ductility Class E reinforcement; and
 - (iii) structures with an importance level 4 shall be designed to remain serviceable for immediate use following a design event for importance level 2 structures (refer to AS 1170.4).

14.2 DEFINITIONS

For the purpose of this Standard, the definitions given in AS 1170.4 and those below apply. Where the definitions in this Standard differ from those given in AS 1170.4, for the purpose of this Standard, those below apply.

NOTE: A number of definitions given in AS 1170.4 are repeated here to avoid users having to refer to AS 1170.4. Those marked with an asterisk (*) have been modified from those in AS 1170.4 to suit their application in this section.

14.2.1 Connection

Mechanical means that provide a load path for actions between structural elements, non-structural elements and structural and non-structural elements.

14.2.2 Ductility (of a structure)

Ability of a structure to sustain its load-carrying capacity and dissipate energy when responding to cyclic displacements in the inelastic range during an earthquake.

14.2.3 Earthquake actions

Inertia-induced actions arising from the response to earthquake of the structure.

14.2.4 Intermediate Moment-resisting Frame (IMRF)

Concrete moment-resisting frame designed and detailed in accordance with this Standard to achieve moderate structural ductility (see Table 14.3) and which conforms with the specific earthquake detailing requirements of this section (Section 14).

14.2.5 Moment-resisting frame

Essentially complete space frame that supports the vertical and horizontal actions by both flexural and axial resistance of its members and connections.

14.2.6 Ordinary Moment-resisting Frame (OMRF)

Moment-resisting frame with no particular earthquake detailing except for columns (see Clause 14.4, as specified in this Standard (see Table 14.3).

14.2.7 Space frame

A three-dimensional structural system composed of interconnected members, other than loadbearing walls, which is capable of supporting vertical loads and may also provide horizontal resistance to earthquake forces.

14.2.8 Structural ductility factor (μ)

Numerical assessment of the ability of a structure to sustain cyclic displacements in the inelastic range. Its value depends upon the structural form, the ductility of the materials and structural damping characteristics.

14.2.9 Structural performance factor (S_p)

Numerical assessment of the additional ability of the total building (structure and other parts) to survive earthquake motion.

14.2.10 Structural wall

A2 | Walls (either loadbearing or non-loadbearing) connected to floor and/or roof diaphragms that attract horizontal earthquake and wind design actions.

Structural walls may be considered as Non-Ductile, Limited Ductile, or Moderately Ductile and analysed in accordance with AS 1170.4 using the structural ductility and structural performance factors nominated in Table 14.3.

A2 | Short or squat walls, as defined in Clause 14.4.4.4, shall be treated as non-ductile walls in accordance with Clause 14.3.

14.3 STRUCTURAL DUCTILITY FACTOR (μ) AND STRUCTURAL PERFORMANCE FACTOR (S_p)

The structural ductility factor (μ) and the structural performance factor (S_p) for concrete structures and members constructed in accordance with this Standard shall be as given in Table 14.3.

TABLE 14.3
STRUCTURAL DUCTILITY FACTOR (μ) AND STRUCTURAL
PERFORMANCE FACTOR (S_p)

Structural system description	(μ)	S_p	S_p/μ	μ/S_p
Special moment-resisting frames (fully ductile) designed in accordance with NZS 1170.5 and NZS 3101 and the AS 1170.4 Hazard Map	4	0.67	0.17	6
Ductile structural walls designed in accordance with NZS 1170.5 and NZS 3101 and the AS 1170.4 Hazard Map	4	0.67	0.17	6
Ductile partially or fully coupled walls designed in accordance with NZS 1170.5 and NZS 3101 and the AS 1170.4 Hazard Map	4	0.67	0.17	6
Intermediate moment-resisting frames (moderately ductile) designed in accordance with Section 2.2 of this Standard and Clauses 14.4 and 14.5 of this section	3	0.67	0.22	4.5
Combined systems of intermediate moment-resisting frames and moderately ductile structural walls designed in accordance with Section 2.2 of this Standard and Clauses 14.4, 14.5 and 14.7 of this section	3	0.67	0.22	4.5
Moderately ductile structural walls designed in accordance with Section 2.2 of this Standard and Clauses 14.4 and 14.7 of this section	3	0.67	0.22	4.5
Ordinary moment-resisting frames designed in accordance with Section 2.2 of this Standard and Clause 14.4 of this section	2	0.77	0.38	2.6
Ordinary moment-resisting frames in combination with limited ductile shear walls designed in accordance with Section 2.2 of this Standard and Clauses 14.4 and 14.6 of this section	2	0.77	0.38	2.6
Limited ductile structural walls designed in accordance with Section 2.2 of this Standard and Clauses 14.4 and 14.6 of this section	2	0.77	0.38	2.6
Non-ductile structural walls designed in accordance with Section 2.2 of this Standard and Clause 14.4 of this section	1	0.77	0.77	1.3

Structures and systems not covered in the above table shall have structural ductility factor (μ) and structural performance factor (S_p) determined by a rational analysis. Where combined systems are used, the values of these factors shall be those required for the least ductile elements in the combined system. Design shall be in accordance with Section 2.2 of this Standard and the relevant clauses in Section 14 for each element type.

14.4 GENERAL EARTHQUAKE DESIGN REQUIREMENTS

14.4.1 Design for strength

All members shall be designed for strength for the earthquake loads as defined in AS 1170.4 in accordance with this section and Clause 2.2.

14.4.2 Inter-storey drift

All vertical load-bearing elements shall be designed for the calculated horizontal drifts from earthquake actions as defined in AS 1170.4.

Prefabricated concrete members and other non-structural elements which are attached to, supported by, or enclose the exterior of a structure shall be capable of accommodating movements of the structure resulting from the relevant earthquake actions as follows:

- (a) All connections and panel joints shall allow for the expected relative movement between floors in adjoining storeys.
- (b) Connections shall have sufficient ductility and rotational capacity to preclude non-ductile failure of the connection.

14.4.3 Moment resisting frames

For an ordinary moment frame forming part of the seismic-force-resisting system, beams shall have at least two of the longitudinal bars continuous along both the top and bottom faces. These bars shall be fully developed at the face of the supports.

For SFRC members, the tensile reinforcement ratio for steel reinforcement and tendons shall not be less than 0.004.

NOTE: This limitation applies to SFRC members only. It is an interim requirement while further evidence is established for SFRC members with low volumes of conventional reinforcement. This will be addressed in the future review of the standard.

Any column in an Ordinary Moment Resisting Frame (OMRF) which is part of the lateral force resisting system and where the unsupported length $L_u \leq 5D$ where D is the dimension of the column in the direction of the span, shall be detailed in accordance with Clauses 14.5.4 and 14.5.5.

14.4.4 Structural walls

14.4.4.1 General

Walls shall be designed in accordance with Section 10 or Section 11 as appropriate except that the simplified design method for walls subjected to vertical compression forces provided in Clause 11.5 of this Standard shall only be used for non-ductile walls.

Limited ductile structural walls shall also conform with Clause 14.6 and moderately ductile structural walls shall also conform with Clause 14.7.

14.4.4.2 Groups of walls

When two or more walls are directly inter-connected, or linked to each other by coupling beams or the like, the distribution of in-plane load effects between the walls shall be obtained by linear elastic analysis of the overall structure under the global vertical and lateral loads. The analysis shall distribute in-plane load effects to the individual walls in proportion to their relative stiffness, as determined by their gross cross-sectional properties.

The interconnected vertical edges of walls shall be designed to withstand the transmitted vertical shear forces.

14.4.4.3 Axial load limit for elements with $\mu > 1$

All structural walls shall be proportioned to ensure that the vertical stress, N^*/A_g , is less than $0.2 f'_c$, where N^* is the sum of the Seismic Weights defined in AS 1170.4, Clause 6.2.2 acting on the wall.

Increased axial loads resulting from vertical ground accelerations (if appropriate) or system behaviour of the overall structure (e.g. frame action) should be included in N^* .

14.4.4.4 Aspect ratio

Short or squat walls with aspect ratios less than two shall be designed as non-ductile structural walls using the strut and tie methods in Section 12 of this Standard.

14.4.5 Diaphragms

14.4.5.1 General

Diaphragms shall be regarded as non-ductile elements, and shall be designed for seismic actions in accordance with Section 15.

14.4.5.2 Calculation of inertia forces

Inertia forces acting on the diaphragm at a given floor level shall be determined by equivalent static analysis in accordance with AS 1170.4, with diaphragm forces calculated based on a structural ductility factor (μ) equal to 1.0, and a structural performance factor (S_p) of 0.77.

For lower floors in multi-storey buildings, inertia forces shall incorporate higher mode effects to ensure the maximum inertia force is calculated for the floor under design. Alternatively, the inertia force for any floor may be determined by multiplying the maximum seismic distribution factor for the structure at any floor calculated in accordance with AS 1170.4 by the weight of the floor under calculation.

For seismic design, the mass shall be assumed to be distributed evenly over the diaphragm when undertaking seismic load calculations.

14.4.6 Ductility of reinforced and prestressed concrete beams and band beams with $\mu > 1.25$ and $\mu \leq 3$

In all potential plastic hinge zones, except where the compression zone is a cast-in-place flange of a T-beam or L-beam:

- (a) Compression face reinforcement equivalent to 1/3 of the ultimate capacity of the tension face reinforcement and tendons shall be provided.
- (b) The neutral axis depth k_{uo} shall be limited to a maximum of 0.25.

14.4.7 Robustness and structural integrity

Robustness and structural integrity should be checked in accordance with Clause 2.1.3.

Stairs and ramps shall be designed to be serviceable under the maximum design earthquake actions.

14.5 INTERMEDIATE MOMENT-RESISTING FRAMES (IMRFs)

14.5.1 General

Reinforced IMRFs and prestressed IMRFs shall be regarded as moderately ductile if, in addition to the detailing requirements of this Standard, they satisfy the detailing requirements of this clause and Clauses 14.5.2 to 14.5.6, and provided only Ductility Class N steel or prestressing tendons are used as flexural reinforcement.

For SFRC members, the tensile reinforcement ratio for steel reinforcement and tendons shall not be less than 0.006.

Non-flexural elements may be incorporated into a moment-resisting frame provided it is shown that the action or failure of these elements will not impair the capacity of the frame to resist horizontal or vertical forces.

In prestressed members, the quantity of tensile steel (reinforcement and tendons) shall be such that the flexural strength of any section is greater than $1.1(M_{uo})_{\min}$ at that section.

14.5.2 Beams

14.5.2.1 Longitudinal reinforcement

Beams shall be provided with longitudinal reinforcement and tendons as follows:

- (a) The top and bottom face of the beam shall be continuously reinforced.
- (b) The area of reinforcement and tendons provided in a span shall be such that—
 - (i) the positive-moment strength at a support face is not less than one third of the negative-moment strength provided at that face of the support; and
 - (ii) neither the negative nor the positive-moment strength at any section along the member length is less than one fifth of the maximum moment strength provided at the face of either support.

- (c) Longitudinal reinforcement shall be continuous through intermediate supports. When framing into external columns, the longitudinal reinforcement shall be extended to the far face of the confined region and anchored to develop the yield strength of the reinforcement at the span face of the support.
- (d) Lapped splices in longitudinal reinforcement, located in a region of tension or reversing stress, shall be confined by at least two closed ties at each splice.

14.5.2.2 Shear reinforcement

Beams shall be provided with shear reinforcement conforming with the following requirements as well as the requirements of Sections 8.2 and 8.3:

- (a) The design shear force at any cross-section shall be at least the lesser of:
 - (i) The sum of the shear associated with development of nominal moment strengths ($\phi = 1.0$) of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity loads (see Figure 14.5.2.2).
 - (ii) The maximum shear obtained from design load combinations that include earthquake action E, with E taken as twice that prescribed by AS 1170.4.
- (b) Shear reinforcement shall be perpendicular to the longitudinal reinforcement; be provided throughout the length of the member; have at least two legs; and have a maximum spacing of $0.5D$.
- (c) The area of shear reinforcement (A_{sv}) shall be not less than $0.5b_v s/f_{sy.f}$.
- (d) Over a distance of at least $2D$ from the face of a support, shear reinforcement shall be closed ties, with the first tie located 50 mm from the support face. These closed ties shall be spaced at centres not greater than $0.25 d_o$, $8 d_b$, $24 d_f$ or 300 mm, whichever is least,

where

d_b = diameter of the smallest longitudinal bar enclosed by the tie; and

d_f = diameter of the bar forming the tie.

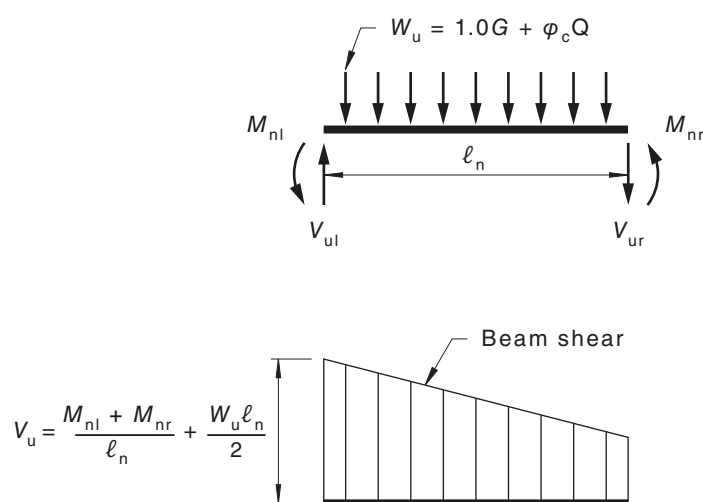


FIGURE 14.5.2.2 SHEAR FROM GRAVITY LOADS AND REVERSE CURVATURE BENDING

14.5.3 Slabs

14.5.3.1 General

Slabs shall conform with Paragraph 14.5.2.1, Items (a), (b) and (c). Two-way flat slabs forming part of a moment-resisting frame shall also conform with Paragraph 14.5.3.2.

14.5.3.2 Reinforcement detailing in flat slabs

Reinforcement in flat slabs shall be located and anchored in accordance with the following:

- (a) The top and bottom faces of the column strips shall be continuously reinforced in both directions.
- (b) All reinforcement, which is provided to resist the portion of the slab moment transferred to the support, shall be placed within the column-strip defined in Clause 6.1.4.1.
- (c) A proportion of the reinforcement required by Item (a) above shall be evenly distributed in a width of slab between planes that are 1.5 times the thickness of the slab or drop panel beyond faces of the column or capital.

The proportion to be distributed is given by—

$$1/\left\{1+(2/3)\left[\sqrt{(b_l + d_o)/(b_t + d_o)}\right]\right\} \quad \dots 14.5.3.2$$

or 0.5, whichever is greater,

where

b_l = size of rectangular, or equivalent rectangular, column, capital, or bracket, measured in the direction of the span for which moments are being determined

b_t = size of rectangular, or equivalent rectangular, column, capital, or bracket, measured transverse to the direction of the span for which moments are being determined

- (d) Negative moment strength at any section in the column strip along the member length shall not be less than one quarter of the maximum moment strength provided at the face of either support. Not less than one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span.
- (e) Positive moment strength at any point in the column strip shall be not less than one third of the maximum negative moment strength provided at the face of either support, nor less than half of the maximum positive moment strength provided in the span.
- (f) At discontinuous edges of the slab, all top and bottom reinforcement at a support shall be capable of developing its yield strength at the face of the support.

14.5.4 Columns

At each end of the clear height of a column within a storey, the longitudinal reinforcement shall be restrained by closed fitments for a distance from the end equal to the greater of the maximum dimension of the column cross-section, or one sixth of the least clear height between consecutive flexural members framing into it.

The spacing of the closed fitments shall not be greater than required by Clauses 10.7.3 and 10.7.4 and the shear requirements of Clause 14.5.2.2(b), (c) and (d) and the smallest of the following limits:

A2

(a) 8 times the diameter of the smallest longitudinal bar enclosed.

A2 (b) $24 \sqrt{\left(\frac{f_{sy,f}}{500}\right)}$ times the diameter of the fitment bar

(c) One-half of the smallest cross-sectional dimension of the column.

(d) 300 mm.

A2 The first fitment shall be located at half of this spacing from the face of the relevant support.

The cross-sectional area of the fitments shall be provided to satisfy the shear requirements for the column but not less than that required by Clauses 10.7.3 and 10.7.4.

Where

A2 $N^* > \phi 0.3 A_g f'_c$ or $f'_c > 65 \text{ MPa}$,

each longitudinal bar shall be restrained by a closed fitment.

14.5.5 Column joints

Joints between columns and flexural members framing into them shall be confined by closed ties throughout the depth of the joint.

A2 The spacing of the closed fitments shall be not greater than required by Clause 14.5.4, except that, where a slab or beams exist on all four sides of the column, the cross-sectional area may be reduced by half this value for the depth of the shallowest slab or beam.

14.5.6 Robustness and structural integrity

In structures where a moment resisting frame is being relied on to act as part of the lateral support system in an earthquake, to reduce the likelihood of column failure under earthquake loading and to promote a strong column/weak beam strength hierarchy at the joint, M_{nc} is required to be $> 6/5 M_{nb}$. That is, the sum of the nominal flexural strengths of columns (calculated in conjunction with the applied axial loads due to earthquake design actions) framing into the joint (evaluated at the faces of the joint) shall be greater than or equal to 1.2 times the sum of nominal flexural strengths of the beams framing into the joint (evaluated at the faces of the joint).

If columns cannot conform with these requirements, then they cannot be considered as part of the lateral support system in an earthquake. Such columns elements must however be designed for drift induced moments arising from frame action.

14.6 LIMITED DUCTILE STRUCTURAL WALLS

14.6.1 General

Vertical and horizontal reinforcement shall be provided on both wall faces and shall be divided equally between the two wall faces.

14.6.2 Boundary elements

14.6.2.1 General

Where required by this clause, structural walls shall be provided with boundary elements in accordance with Clause 14.6.2.2 or 14.6.2.3 depending on building height.

In any storey, boundary elements shall be provided at discontinuous edges of structural walls and around openings through them if—

(a) the vertical reinforcement within the storey height is not laterally restrained in accordance with Clause 10.7.4; and

- (b) the calculated extreme fibre compressive stress in the wall exceeds $0.15 f'_c$.

The stress referred to in Item (b) shall be calculated using the design action effects for the strength limit state, a linear-elastic strength model and the gross cross-section properties of the wall.

Boundary elements are not required around openings located within the middle third of a wall that are less than 25% of the wall effective height and 25% of the overall wall length.

14.6.2.2 Structures of not more than four storeys

For structures of not more than four storeys above their structural base and where boundary elements are required, an integrally cast column, or additional edge reinforcement consisting of two N16 bars tied with an N12 U-bar or four N12 bars enclosed with R10 closed stirrups as shown in Figure 14.6.2.2 below, shall be deemed to satisfy this requirement. Ties shall be spaced at the lesser of t_w and 200 mm.

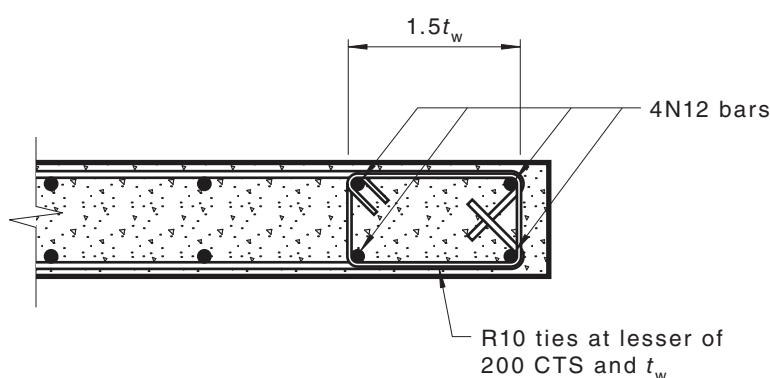


FIGURE 14.6.2.2 REINFORCEMENT OF BOUNDARY ELEMENTS FOR BUILDINGS OF NOT MORE THAN FOUR STOREYS

14.6.2.3 Structures of more than four storeys

Where boundary elements are required, the horizontal cross-section of the wall shall be treated as an I-beam in which the boundary elements are the flanges and the section of wall between them is the web. Restraint of the longitudinal reinforcement in boundary elements shall conform with Clause 10.7.4 of this Standard or, if the extreme fibre compressive stress calculated as above exceeds $0.2 f'_c$, with Clause 14.5.4 of this Standard. The longitudinal reinforcement shall be restrained in any area of the wall where the compression stress in the wall as calculated above is greater than $0.15 f'_c$.

The boundary elements shall be restrained by closed fitments nominally horizontal at not greater than the lesser of t_w and 200 mm centres vertically, anchored around the edge reinforcement as shown in Figure 14.6.2.3 below.

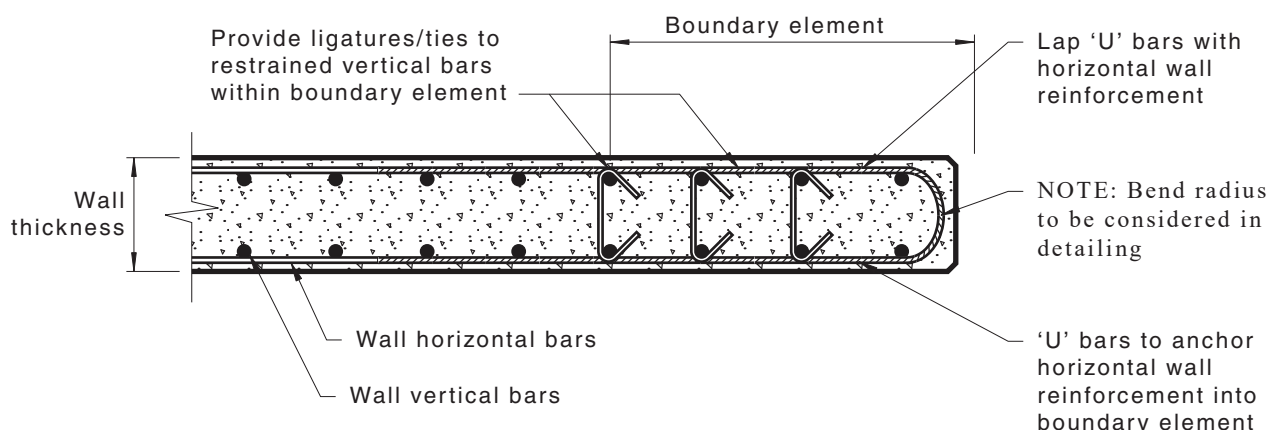


FIGURE 14.6.2.3 REINFORCEMENT OF BOUNDARY ELEMENTS FOR BUILDINGS GREATER THAN FOUR STOREYS

14.6.3 Confinement of the wall core

For structural walls where $f'_c > 50$ MPa confinement of the wall core shall be provided throughout by fitments in accordance with Clause 14.5.4.

14.6.4 Mean concrete strength

The mean value of the 28 day cylinder strength as tested shall be less than $1.4 f'_c$.

14.6.5 Effective height to thickness

The ratio of the effective height to thickness shall be equal to or less than 20.

14.6.6 In-plane shear

In determining the required shear strength of a structural wall and supporting foundations under earthquake design actions, allowance for the effects of flexural over-strength and dynamic amplification shall be included. This requirement is satisfied when the wall shear capacity (ϕV_u) at a cross-section is not less than:

$$\phi V_u \geq \min \left[\left(\frac{1.6 M_u}{M^*} \right) V^*; \left(\frac{\mu}{S_p} \right) V^* \right] \quad \dots 14.6.6$$

14.6.7 Reinforcement

The minimum vertical reinforcement ratio (ρ_{wv}) shall not be less than 0.0025. In plastic hinge regions this shall be increased to—

(a) $\rho_{wv} > 0.7(f'_c)^{0.5}/f_{sy}$ in Critical Tension Reinforcement Zones as follows:

- (i) The outermost L_w region at each free end of a wall in inplane bending as shown in Figure 14.6.7(A), where L_w is defined below.

or

- (ii) An integrated column at the end of the wall with a minimum area of $L_w \times t_w$ as shown in Figure 14.6.7(B).

or

- (iii) Full length of a transverse wall in an interconnected group of walls (e.g. core wall) where the wall is at the extreme tension face as shown in Figure 14.6.7(C).

(b) $\rho_{wv} > 0.35(f'_c)^{0.5}/f_{sy}$ in the remainder of the wall cross-section.

- (c) This increased minimum reinforcement ratio shall extend vertically from the base of the wall for a distance of not less than—
- (i) the greater of twice the wall length ($2L_w$); or
 - (ii) the height of the lower two storeys.

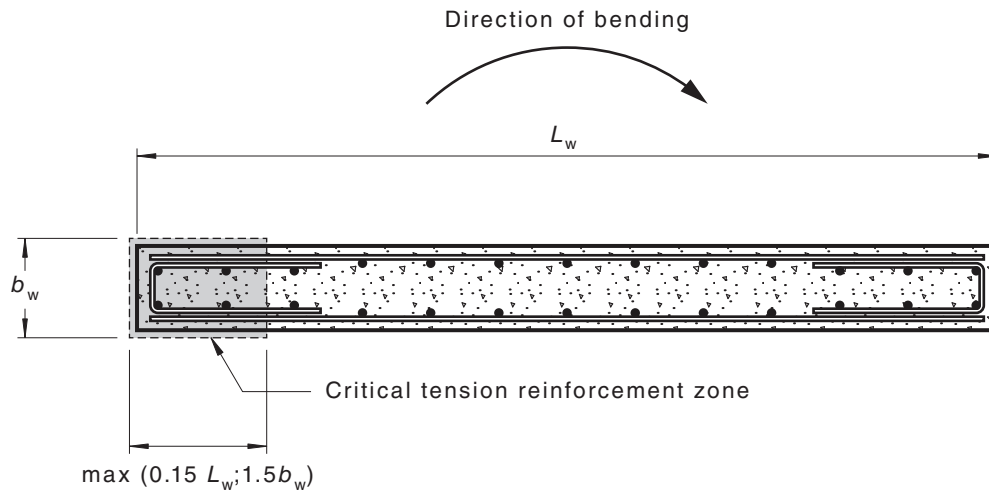


FIGURE 14.6.7(A) CRITICAL TENSION REINFORCEMENT ZONE AT FREE END OF WALL

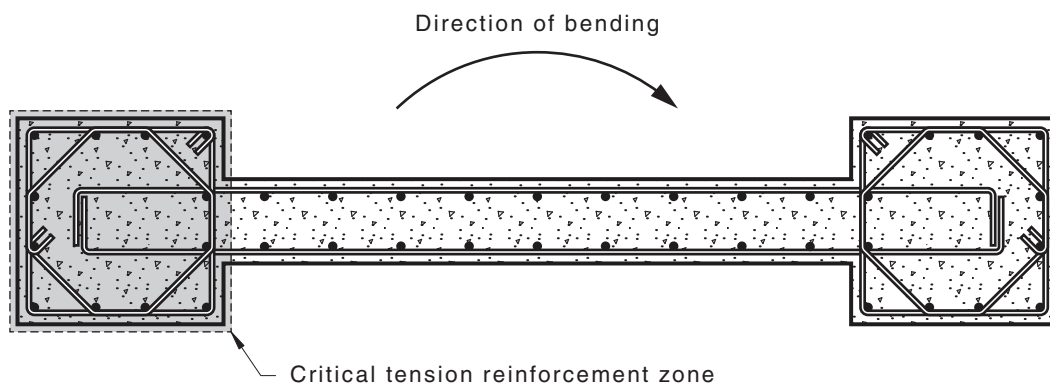


FIGURE 14.6.7(B) CRITICAL TENSION REINFORCEMENT ZONE AT THE END OF A WALL INTEGRATED WITH A COLUMN

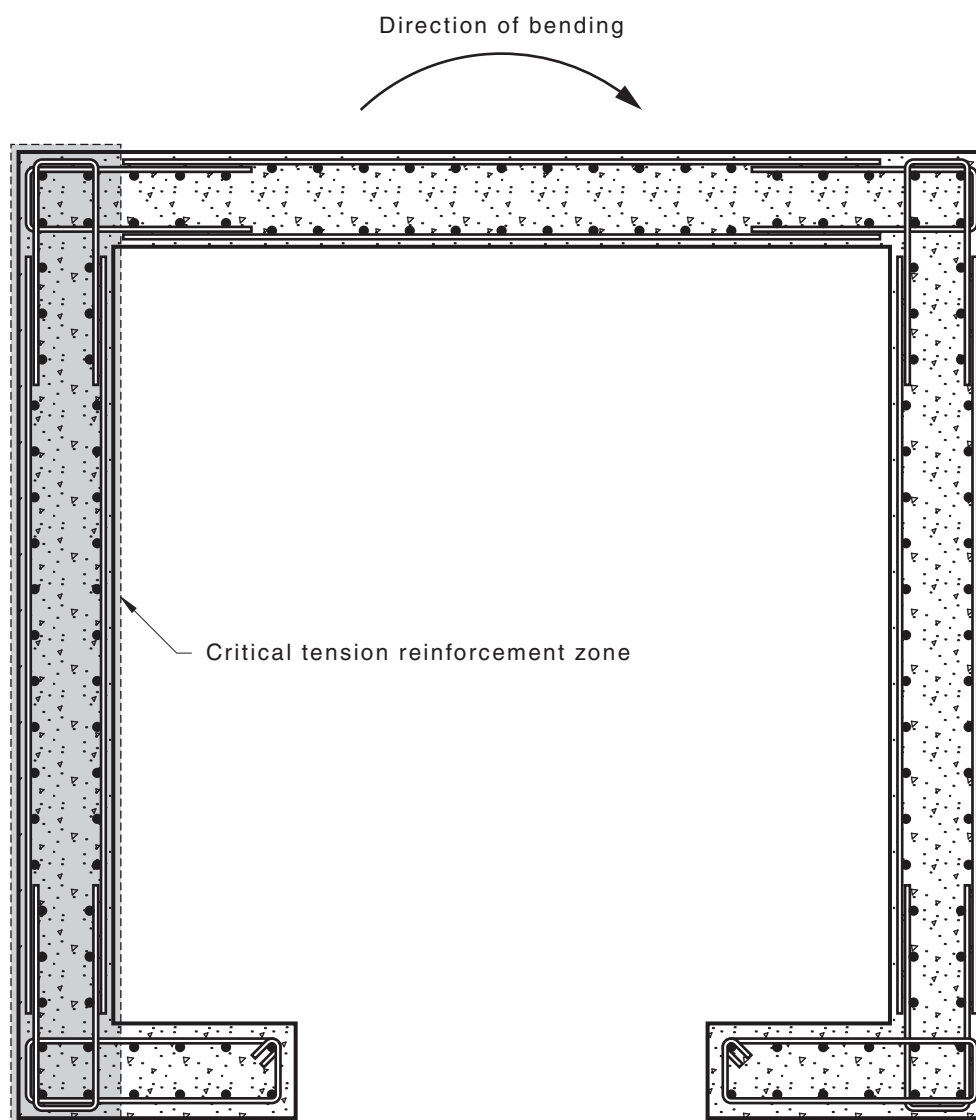


FIGURE 14.6.7(C) CRITICAL TENSION REINFORCEMENT ZONE
FOR AN INTERCONNECTED GROUP OF WALLS

The length at the end of the wall L_w requiring concentrated reinforcing for ductility shall be taken as the greater of $0.15L_w$ and $1.5 t_w$.

For interconnected groups of structural walls the longest wall of the group shall be used in determining the height over which the minimum vertical reinforcement ratio shall apply.

A2 | The minimum vertical reinforcement ratio may be reduced by 10% per floor above the height defined in Clause 14.6.7(c) to a minimum value of 0.0025.

The vertical reinforcement ratio (ρ_{wv}) shall not exceed $16/f_{sy}$ except in regions where lapped splices in boundary elements are unavoidable, in which case the total ratio including the increased area of reinforcement at splices shall not exceed $21/f_{sy}$.

The horizontal reinforcement ratio (ρ_{wh}) shall not be not less than 0.0025.

Vertical and horizontal reinforcement shall be provided on both wall faces and shall be divided equally between the two wall faces and detailed as follows:

- A2 | (i) The ends of all horizontal lapped bars within the central two-thirds region of the wall (i.e. the web region) shall be provided with minimum 135° hooks and lapped with a full-strength splice [see Figure 14.6.7(D)].

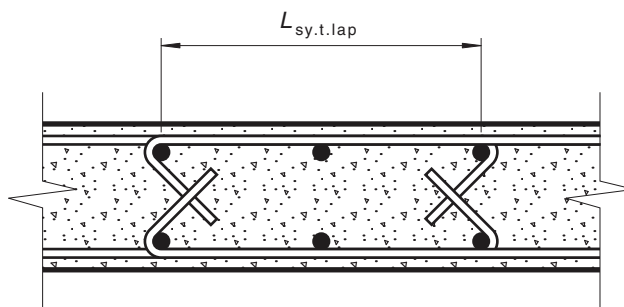


FIGURE 14.6.7(D) HORIZONTAL WALL BAR LAP DETAIL

A2

- (ii) At the ends of walls, where boundary elements exist, horizontal bars shall be provided with hooks or cogs and fully developed and anchored into the confined concrete core of the boundary element or lapped with U bars with a lap length equal to $1.2L_{sy.t.lap}$. Where no boundary element is required, or the U bar provisions of Clause 14.6.2.2 apply, ends of horizontal bars shall be terminated at the end of the wall with full tension laps with U bars of the same diameter.
- (iii) Ductility Class L reinforcement shall not be used as structural reinforcement.

Wall reinforcement terminating at the bottom of the wall in footings, columns, slabs, or beams shall be anchored to develop the yield stress in the reinforcement at the junction of the wall with the terminating member.

14.7 MODERATELY DUCTILE STRUCTURAL WALLS

14.7.1 General

Moderately ductile structural walls shall conform with Clauses 14.6 except—

- (a) Clause 14.6.5; and
- (b) in Clause 14.6.2 all vertical reinforcement in boundary elements shall be restrained in accordance with Clause 14.5.4 irrespective of the calculated compressive stress.

14.7.2 Effective height to thickness

The ratio of the effective height to thickness shall be equal to or less than 16.

14.7.3 Vertical reinforcement laps

In the region extending for a vertically from the base of the wall immediately above the footing to a height equal to—

- (a) the greater of twice the wall length; and
- (b) the height of the lower two storeys.

Laps or mechanical splices to vertical reinforcement shall be evenly staggered along the length of the wall such that, at every horizontal section, not more than 50% of the vertical reinforcement is lapped or spliced. Staggered lapped splices shall conform with the arrangement shown in Figure 13.2.2.

For interconnected groups of structural walls the longest wall of the group shall be used in determining the height over which this clause shall apply.

SECTION 15 DIAPHRAGMS

15.1 GENERAL

This section applies to the design of both conventionally reinforced and prestressed diaphragms, including:

- (a) Cast in situ slabs, with or without beams.
- (b) Cast in situ topping slabs on prefabricated concrete elements.

Diaphragms shall be considered as part of the primary structure with identifiable internal load paths to transfer the required actions between the diaphragms and associated connected elements to the elements of the lateral force resisting structural system.

Elements within diaphragms and connections between diaphragms and elements of the lateral force resisting structural system shall be capable of accommodating both the imposed displacement and force demands imposed on them.

15.2 DESIGN ACTIONS

15.2.1 General design actions

Design actions for diaphragms shall include actions arising from all of the following loadings:

- (a) Associated permanent (dead load) and imposed (live load) actions on the floor or roof.
- (b) Diaphragm in-plane forces due to lateral loads acting on the building, including floor design accelerations and associated inertia loads under seismic actions (see Clause 14.4.5).
- (c) Force transfer between the lateral force resisting elements interconnected by the diaphragm.
- (d) Interaction with elements vertically or near vertically supporting the diaphragm.

Load combinations for diaphragms shall be as provided in AS 1170.0. Where required, it shall be assumed that in-plane and out-of-plane loadings occur concurrently on the diaphragm.

Forces induced in the diaphragm by the lateral load resisting system, including at discontinuities in the vertical stiffness of the building and plan irregularity between stories, shall be considered in design.

15.2.2 Analysis procedure

15.2.2.1 General

Rational analysis shall be used to establish that there is adequate in-plane flexural and shear strength at the ultimate limit state. The analysis may be based on a strut and tie model of the diaphragm in accordance with Section 7. The effect of openings and penetrations shall be considered in analysis.

The effect of cracking and joint opening due to distributed reinforcement throughout the diaphragm resisting in-plane tension forces shall be determined in analysis of the diaphragm. Alternatively, these effects may be ignored if the diaphragm reinforcement resisting in-plane moments is distributed within one quarter of the width of the diaphragm from the tension edge of the diaphragm.

15.2.2.2 *Stiffness*

Analysis for the internal forces transmitted between diaphragms and their supports shall account for the stiffness of the chosen load path.

Where the maximum lateral deformation of the diaphragm is calculated to be more than half the average inter-story deflection of the vertical lateral force resisting elements of the associated storey the diaphragm shall be considered to be flexible, and the displacement of the diaphragm shall be considered in the analysis of the structure.

15.3 CAST IN-PLACE TOPPINGS

A topping slab cast in place on a prefabricated concrete floor or roof shall be permitted to be used as a structural diaphragm, provided that—

- (a) the cast in-place topping acting alone is proportioned and detailed to resist the design diaphragm forces, and the topping has a minimum thickness of 75 mm; or
- (b) sufficient reinforcement is provided in accordance with Clause 8.4, is provided to ensure the topping slab and precast elements act as a composite unit; and
 - (i) the surface of the previously hardened concrete on which the topping slab is placed is clean, free of laitance, and intentionally roughened; and
 - (ii) the topping slab has a minimum thickness of 65 mm excluding the thickness of the prefabricated concrete element at all locations within the diaphragm.

15.4 DIAPHRAGM REINFORCEMENT

15.4.1 General

Where concentrated actions are developed in the diaphragm (such as within collector elements, and at the perimeter of the diaphragm under in-plane bending) Ductility Class N reinforcement or bonded post-tensioning shall be used to resist diaphragm forces.

Reinforcement designed to resist diaphragm in-plane forces shall be in addition to reinforcement provided to resist other load effects, except reinforcement provided to resist shrinkage and temperature effects shall be permitted to also resist diaphragm forces.

15.4.2 Minimum reinforcement

Reinforcement shall be placed in both orthogonal directions, with minimum reinforcement ratios provided in accordance with Clause 9.5.3. The maximum spacing of reinforcement shall not exceed the requirements of Clause 9.5.1.

15.4.3 Effect of development and laps

The effect of development and laps shall be considered in the capacity of the reinforcement at all locations within the diaphragm in accordance with Section 13. Stresses induced in reinforcement from vertical actions shall be included when calculating reinforcement requirements.

15.4.4 Collectors

Collectors shall be provided to transfer concentrated diaphragm forces between the diaphragm and the vertical elements of the lateral load resisting system.

Collectors shall extend from the vertical element of the lateral load resisting system a minimum of one development length beyond the point at which the analysis determines the collector element is no longer in tension.

Collectors shall extend along the vertical element for the larger of either—

- (a) the length required to fully develop the reinforcement in tension at the face of the wall determined in accordance with Section 13;

- (b) the length required to transmit the design force from the collector into the vertical element through longitudinal shear determined in accordance with Section 8.4; or
- (c) mechanical or tension laps with horizontal reinforcement determined in accordance with Section 13.

Splices shall only be permitted in collector elements where the splice length is sufficient to develop the full yield strength of the reinforcement in accordance with Section 13.

15.4.5 Construction joints

Transfer of diaphragm forces across construction joints shall only be assumed where sufficient fully developed reinforcement is placed across the joint to transmit the calculated diaphragm forces.

Mechanical connectors may be used to transfer diaphragm forces across joints, or between prefabricated concrete and in situ elements in a prefabricated concrete topping, provided the connectors are designed to resist the required tension under the anticipated joint opening.

SECTION 16 STEEL FIBRE REINFORCED CONCRETE

16.1 GENERAL

This section sets out minimum requirements for building structures and members that contain steel fibres conforming to either ISO 13270 or EN 14889. The design rules are not intended to be used for slabs on grade, or for temporary sprayed concrete linings, or to design applications in which increased resistance to plastic shrinkage, increased resistance to abrasion or impact are aimed for.

The design of steel fibre reinforced concrete (SFRC) at both the ultimate and serviceability limit states shall be based on appropriate stress (σ)–COD (w) relationships for SFRC. The stress (σ)–COD (w) relationships for SFRC in tension shall be determined by testing as specified in Clause 16.3.3.

Steel fibres may be used in frames or suspended beams and slabs in addition to conventional reinforcement or tendons. Steel fibres may be used for statically indeterminate systems where it can be shown that cracking would not cause sudden collapse.

Redistribution of moments and plastic methods of analysis may be used for the strength design of continuous beams and frames in accordance with Clause 2.2.2, provided it is shown that high moment regions possess sufficient moment-rotation capacity to achieve the plastic redistribution implied in the analysis.

Steel fibres shall not be relied upon at construction joints for either serviceability or strength requirements.

Design procedures in this section are for steel-fibre-reinforced concrete with a softening classification only (see Figure 16.3.3.1). Hardening SFRC and the use of synthetic fibres are beyond the scope of this Standard.

NOTES:

- 1 When using brittle fibres that rely on a fibre pullout failure mechanism to obtain member ductility, care is required for cases where a significant number of fibres fracture or where fibres result in local crushing of the concrete due to the local forces imposed on the matrix by the fibres. Fibre fracture may occur where the bond between the fibre and the matrix is high, and this is more likely in a high strength concrete combined with fibres of high bond capacity and of lower strength steels. In assessing the suitability of a given fibre for a given application, bond-strength gain in time and the resulting potential loss of ductility should be considered.
- 2 The rules of this section meet the performance requirements defined by Part A0 of the NCC Building Code of Australia as a ‘deemed-to-satisfy’ solution. Other designs are allowable under NCC provided that the ‘performance solution’ pathway is satisfied.

A2

16.2 DEFINITIONS

For the purposes of this section the definitions below apply.

16.2.1 Crack mouth opening displacement (CMOD)

The width of a crack measured at its mouth in a flexural tensile test undertaken in accordance with EN 14651.

16.2.2 Crack opening displacement (COD)

The width of a single localized crack, taken as an average on four sides, for a direct tensile test on a strain softening SFRC dog-bone shaped specimen in accordance with Appendix C.

16.2.3 Hardening behaviour

A material that displays an enhanced tensile strength with increasing crack widths beyond that measured at the point of initial cracking of the cementitious matrix.

16.2.4 Softening behaviour

A material that exhibits a loss of tensile strength upon cracking.

16.2.5 Steel fibre reinforced concrete (SFRC)

A mixture of concrete and steel fibres.

16.2.6 Target dosage

The specified quantity of fibres in kilograms per cubic metre of concrete (kg/m³).

16.3 PROPERTIES OF SFRC

16.3.1 General

SFRC shall be classified in terms of both its characteristic compressive (cylinder) strength (f'_c) (see Clause 16.3.2) and its characteristic residual tensile strength ($f'_{1.5}$) (see Clause 16.3.3.3).

16.3.2 Compressive strength

The characteristic compressive strength of SFRC at 28 days (f'_c) shall be determined in accordance with Clause 3.1.1.1.

In the absence of more accurate data, the mean value of the in situ compressive strength (f_{cmi}) shall be taken as 90% of the mean value of the cylinder strength (f_{cm}).

NOTE: Alternatively, the characteristic residual tensile strength may be determined by rational calculation where the extreme tensile fibre strain is limited to 0.025, but taken as not greater than the residual tensile strength for a crack width of 1.0 mm.

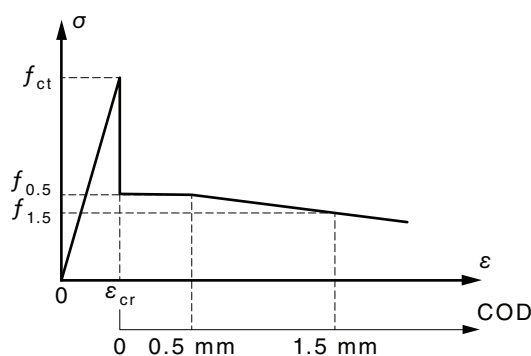
16.3.3 Tensile properties

16.3.3.1 Classification

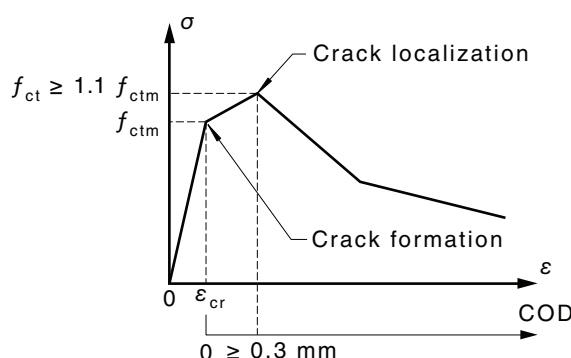
SFRC shall be classified as either softening or hardening as shown in Figure 16.3.3.1.

A hardening SFRC is outside the scope of this section.

NOTE: A hardening material is defined as one with a tensile strength equal to or greater than 1.1 times the strength of the matrix without fibres and taken at a crack opening displacement (COD) of equal to or greater than 0.3 mm [see Figure 16.3.3.1(b)].



(a) Strain softening SFRC



(b) Strain hardening SFRC

FIGURE 16.3.3.1 CLASSIFICATION OF SFRC

16.3.3.2 Matrix tensile strength

The matrix tensile strength of the softening SFRC (f_{ct}) shall be obtained using direct or indirect tensile testing, in accordance with Clause 3.1.1.3.

When only the characteristic compressive (cylinder) strength (f'_c) has been determined, the mean and characteristic flexural tensile strength of SFRC shall be calculated in accordance with Clause 3.1.1.3.

16.3.3.3 Residual tensile strength

The standard characteristic residual tensile strength grades ($f'_{1.5}$) are 0.4 MPa, 0.6 MPa, 0.8 MPa, 1.2 MPa, 1.6 MPa and 2.0 MPa.

The characteristic residual tensile strengths of concrete at 28 days ($f'_{1.5}$) shall be determined statistically from tests carried out in accordance with Clause 16.3.3.4 or Clause 16.3.3.5.

Higher residual tensile strength grades are permitted if supported by direct tensile testing undertaken in accordance with Clause 16.3.3.4.

NOTES:

- 1 Where similar SFRC mixes have been tested in accordance with Clause 16.3.3.5 and the difference in fibre content is not more than 20 kg/m³, the residual tensile strengths that fall between these limits can be obtained by linear interpolation.
- 2 Specification of characteristic residual tensile strengths other than the specified standard strength grades is permitted.

16.3.3.4 Determination of strength by direct testing

The characteristic residual tensile strengths $f'_{0.5}$ and $f'_{1.5}$ shall be obtained using direct tensile tests:

- (a) as specified in Appendix C; or
- (b) by testing using an independently approved and verified testing method.

In (a) and (b), the residual tensile strength results obtained from the test shall be multiplied by the three-dimensional orientation factor k_{3Dt} , where—

$$k_{3Dt} = \frac{1}{0.94 + 0.6l_f / b} \leq 1 \quad \dots 16.3.3.4(1)$$

Alternatively, where matched direct and indirect testing has been undertaken in accordance with Clause 16.3.3.6 for similar SFRC mixtures, the characteristic residual tensile may be determined as—

$$f'_{0.5} = k_{R,2} f'_{R,2} \quad \dots 16.3.3.4(2)$$

$$f'_{1.5} = k_{R,4} f'_{R,4} \quad \dots 16.3.3.4(3)$$

where $f'_{R,2}$ $f'_{R,4}$ are determined in accordance with Clause 16.3.3.7 and calculated statistically, and the factors $k_{R,2}$ and $k_{R,4}$ determined from Clause 16.3.3.6.

For the purposes of this clause, similar SFRC mixtures are defined as having the same—

- (c) fibre type and content;
- (d) water to cementitious material ratio;
- (e) maximum aggregate particle size;
- (f) aggregates of same geological origin; and
- (g) characteristic compressive strength (f'_c).

The characteristic value for the residual tensile strengths shall be determined as follows:

- (i) The strength of the population shall be treated as normally distributed and the characteristic strength determined in accordance with ISO 12491.
- (ii) A confidence level of 75% shall be used such that 95% of the population exceeds the characteristic value.
- (iii) The standard deviation of the population shall be determined from the sample but shall not be taken as less than that calculated for a COV = 0.25.
- (iv) The minimum number of test specimens is six.

The residual tensile stress for other CODs shall be obtained by testing. Alternatively, linear interpolation or extrapolation from a straight line connecting $f'_{0.5}$ and $f'_{1.5}$ may be applied but shall not be taken as greater than $f'_{0.5}$ nor less than zero.

The value of $f'_{1.5}$ shall not be taken as greater than $0.9f'_{0.5}$.

16.3.3.5 Determination of strength by indirect testing

The characteristic residual tensile strengths at 0.5 mm ($f'_{0.5}$) and 1.5 mm COD ($f'_{1.5}$) is obtained using indirect tests as specified in Clause 16.3.3.7 and are calculated as follows:

$$f'_{0.5} = k_{3Db} (-0.04 f'_{R,4} + 0.37 f'_{R,2}) \leq k_{3Db} 0.36 \sqrt{f'_c} \quad \dots 16.3.3.5(1)$$

$$f'_{1.5} = k_{3Db} (0.4 f'_{R,4} - 0.07 f'_{R,2}) \leq k_{3Db} 0.36 \sqrt{f'_c} \quad \dots 16.3.3.5(2)$$

where the three-dimensional orientation factor is—

$$k_{3Db} = \frac{1}{1 + 0.19 l_f / b} \leq 1$$

and b is the prism sectional width.

The value of $f'_{1.5}$ shall not be taken as greater than $0.9 f'_{0.5}$.

The residual tensile stress for other CODs shall be obtained by linear interpolation or extrapolation of Equations 16.3.3.5(1) and 16.3.3.5(2) but shall not be taken as greater than $f'_{0.5}$ nor less than zero.

NOTE: For fibres not greater than 70 mm in length, the three-dimensional orientation factor may conservatively be taken as $k_{3Db} = 0.92$.

16.3.3.6 Residual tensile strength—Residual flexural strength relationship

The relationship between residual tensile strength and the residual flexural strength shall be obtained by matched testing using the same SFRC mixture. Residual tensile strength specimens shall be prepared and tested in accordance with Clause 16.3.3.7. Residual flexural tensile tests shall be in accordance with EN 14651 and—

- (a) the concrete mix shall be batched to ensure a uniform distribution of fibres, the SFRC shall be placed in the moulds in a manner that does not interfere with the distribution of the fibres and, the SFRC shall be compacted using lightly applied external vibration; and
- (b) a minimum of six specimens shall be tested.

The reference factors $k_{R,2}$ and $k_{R,4}$ shall be determined as—

$$k_{R,2} = f'_{0.5m} / f_{R,2m} \quad \dots 16.3.3.6(1)$$

$$k_{R,4} = f'_{1.5m} / f_{R,4m} \quad \dots 16.3.3.6(2)$$

where

$f_{0.5m}$ = mean residual tensile strengths corresponding to a COD of 0.5 mm, determined in accordance with Appendix C

$f_{1.5m}$ = mean residual tensile strengths corresponding to a COD of 1.5 mm, determined in accordance with Appendix C

$f_{R,2m}$ = mean residual flexural tensile strengths corresponding to a CMOD of 1.5 mm, determined in accordance with Clause 16.3.3.7

$f_{R,4m}$ = mean residual flexural tensile strengths corresponding to a CMOD of 3.5 mm, determined in accordance with Clause 16.3.3.7

16.3.3.7 Residual flexural tensile strength

The residual flexural tensile strength ($f_{R,j}$) shall be determined from three point notched bending tests on 150 mm square section prisms. The notch depth shall be 25 mm and the test conducted in accordance with EN 14651. The force F shall be plotted against the crack mouth opening displacement (CMOD), as shown in Figure 16.3.3.7, and the residual flexural stress calculated as—

$$f_{R,j} = \frac{3F_{Rj}L}{2bh_{sp}^2} \quad \dots 16.3.3.7$$

where

b = width of the specimen in millimetres

h_{sp} = distance between tip of the notch and top of cross section in millimetres

L = span in millimetres

F_{Rj} = load recorded at $CMOD_j$ (see Figure 16.3.3.7)

NOTE: Testing should be undertaken in a laboratory accredited by the National Association of Testing Laboratories (NATA).

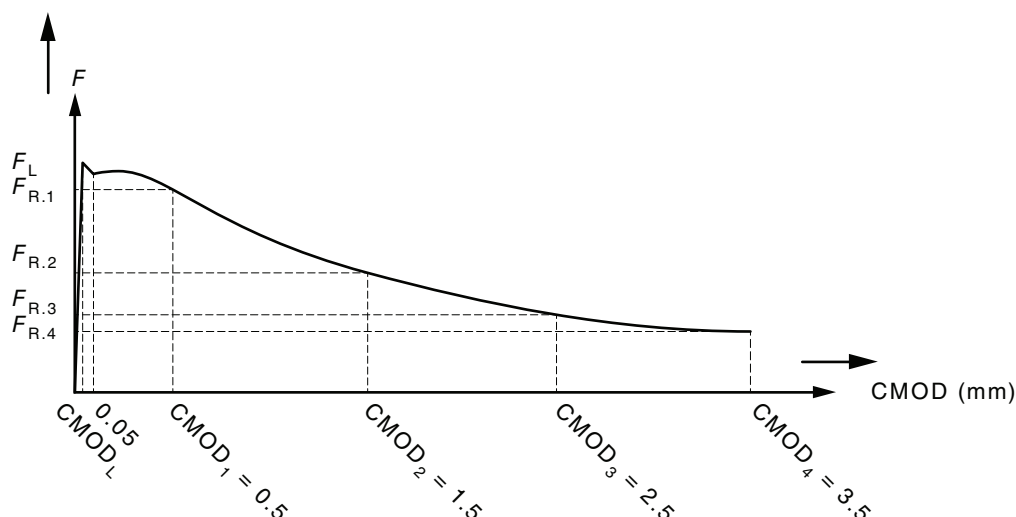


FIGURE 16.3.3.7 LOAD VERSUS CMOD FOR RESIDUAL FLEXURAL TENSION

The characteristic value for the residual flexural tensile strength shall be determined as follows:

- The strength of the population shall be treated as normally distributed and the characteristic strength determined in accordance with ISO 12491.
- A confidence level of 75% shall be used such that 95% of the population exceeds the characteristic value.
- The standard deviation of the population shall be determined from the sample but shall not be taken as less than that calculated for a $COV = 0.25$.
- The minimum number of test specimens is six.

16.3.3.8 Minimum fibres dosage

For serviceability design, a dosage of fibres shall be provided such that the following minimum material performance criterion is achieved—

$$f_{R,1m}/f_{Lm} > 0.4.$$

For strength design, a dosage of fibres shall be provided such that the following minimum material performance criteria are achieved—

- $f_{R,1m}/f_{Lm} > 0.4$; and
- $f_{R,3m}/f_{R,1m} > 0.5$

where

f_{Lm} = mean flexural tensile strength corresponding to the limit of proportionality, determined in accordance with Clause 16.3.3.8 with $F_{R,j}$ substituted by F_L

$f_{R,1m}$ = mean residual flexural tensile strength corresponding to a CMOD of 0.5 mm, determined in accordance with Clause 16.3.3.7

$f_{R,3m}$ = mean residual flexural tensile strength corresponding to a CMOD of 2.5 mm, determined in accordance with Clause 16.3.3.7

Alternatively, the dosage of fibres shall not be less than the greater of—

(i) $12\gamma_s (d_f / l_f)^2$; and

(ii) 20 kg/m^3 ,

where γ_s is the mass density of steel and taken as 7850 kg/m^3 .

16.3.4 Modulus of elasticity

The mean modulus of elasticity at the appropriate age (E_{cj}) shall be determined in accordance with Clause 3.1.2.

16.4 DESIGN OF SFRC MEMBERS CONTAINING REINFORCEMENT OR TENDONS

16.4.1 General

This clause applies to reinforced and prestressed beams that include steel fibres subjected to any combination of shear force, bending moment and axial force.

NOTE: This clause does not apply when torsion acts in conjunction with shear or to non-flexural members.

16.4.2 Strength in bending and combined bending and axial force

Calculations for strength of cross-sections in bending shall incorporate equilibrium and strain-compatibility considerations and be consistent with the following assumptions:

- Plane sections normal to the axis shall remain plane after bending.
- For sections where that part of the cross-section in tension is rectangular, the stress in the SFRC in that part of the cross-section in tension shall be taken to be $0.9k_g f'_{1.5}$, where $f'_{1.5}$ is the characteristic residual tensile stress determined in accordance with Clause 16.3.3.3 and k_g is a factor to take account of member size.
- For I-section members and members where the cross-section in tension is not rectangular, the area of flange overhangs on the tensile side shall be multiplied by a factor of 0.67 for the determination of the internal force on the cross-section provided by fibres.
- The distribution of compressive stress shall be determined from a stress-strain relationship for the concrete in accordance with Clause 3.1.4.
- The maximum strain in the extreme compression fibre is taken as 0.003.

The member size factor is—

$$k_g = 1 + \frac{0.0067 A_{ctu}}{A_o} \leq 1.6 \quad \dots 16.4.2$$

where

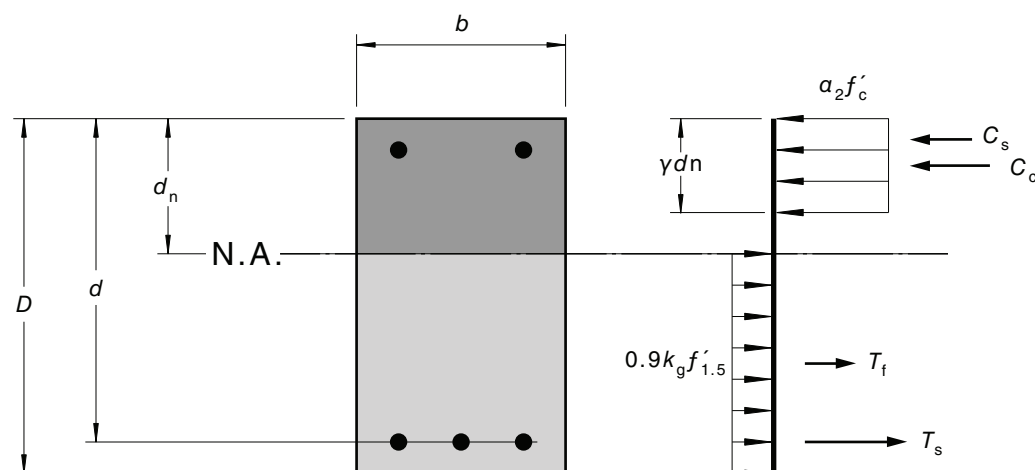
A_{ctu} = area of concrete within the tensile zone (mm^2) at the ultimate limit state

A_o = a reference area taken as $A_o = 15\,600 \text{ mm}^2$.

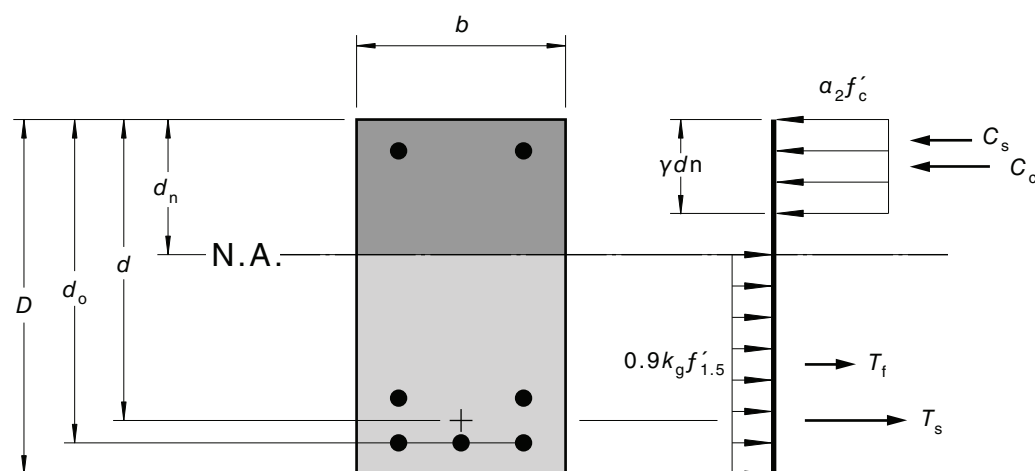
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The strength of a section in bending, or in combined bending and axial force, shall be determined using rectangular stress blocks for the concrete in compression and concrete in tension, as shown in Figure 16.4.2.

In any part of the tensile zone where the average tensile strain ε_t exceeds 0.025, the contribution of the fibres shall be ignored when calculating the strength of a cross-section in bending, where $\varepsilon_t = 0.003(d_z - d_n)/d_n$ and d_z is the depth to a point in the tensile zone.



(a) Single tensile reinforcement layer



(b) Multiple tensile reinforcement layers

FIGURE 16.4.2 STRESS BLOCK AND FORCES ON REINFORCED SFRC SECTION

16.4.3 Minimum reinforcement requirements for bending

The minimum tensile reinforcement shall be not less than that determined using the principles of Clause 8.1.6, excluding fibres.

NOTES:

- 1 This clause does not apply to foundation or pavement slabs fully or partially supported on soil.
- 2 This requirement may be waived at critical sections of a statically indeterminate member, provided it can be demonstrated this will not lead to sudden collapse of a span or a reduced collapse load.

16.4.4 Strength of beams in shear

16.4.4.1 Design shear strength of a beam

The design shear strength of a beam shall be taken as ϕV_u , where—

$$V_u = V_{uc} + V_{uf} + V_{us} \quad \dots 16.4.4.1$$

where V_{uc} and V_{us} are determined from Clause 8.2.3 and V_{uf} is determined from either Clause 16.4.4.2.1 or Clause 16.4.4.2.2.

Notwithstanding Equation 16.4.4.1, the fibres component to the ultimate shear strength of a beam, V_{uf} , shall not exceed the greater of V_{uc} and that determined by Clause 16.4.4.3 with V_{us} taken as zero.

16.4.4.2 Contribution to shear strength by steel fibres

16.4.4.2.1 Design by refined calculation

The contribution of the fibres to the ultimate shear strength (V_{uf}) of an SFRC beam shall be calculated from the following equation:

$$V_{uf} = k_s k_g d_v b_v f'_w \cot \theta_v \quad \dots 16.4.4.2.1(1)$$

where

- A2 | k_s = fibre orientation casting bias factor and is taken as 0.64
 A2 | k_g = member size factor determined by Equation 16.4.2
 f'_w = the characteristic residual tensile strength of SFRC corresponding COD w
 θ_v = angle between the axis of the concrete compression strut and the longitudinal axis of the member calculated in accordance with Clause 8.2.4.2
 d_v = the effective shear depth of the member calculated in accordance with Clause 8.2.1.9

The characteristic residual tensile strength f'_w shall be determined in accordance with either Clauses 16.3.3.4, 16.3.3.5 or 16.3.3.6 where the crack width is determined as—

$$w = (0.2 + 1000 \varepsilon_x) \left(\frac{1000 + k_{dg} d_v}{1300} \right) \frac{1}{\cos \theta} \geq 0.125 \text{ mm} \quad \dots 16.4.4.2.1(2)$$

where ε_x is determined from Clause 8.2.4.3.

Alternatively for beams less than 1000 mm in depth, f'_w in Equation 16.4.4.2.1(1) may be taken as equal to $f'_{1.5}$.

The concrete and steel components shall be determined in accordance with Clauses 8.2.3 with k_v determined from Clause 8.2.4.2.

NOTE: The components V_{uc} , V_{us} and V_{uf} are coupled through the strain parameter ε_x and the equations are solved iteratively.

16.4.4.2.2 Design by simplified calculation

For non-prestressed components not subjected to axial tension, and provided—

- the specified yield strength of the longitudinal reinforcement does not exceed 500 MPa;
- the design concrete strength does not exceed 65 MPa;
- the size of maximum aggregate particle is not less than 10 mm;
- the length of the fibres does not exceed 70 mm; and
- the depth of the beam does not exceed 1000 mm.

The angle of inclination of the compression strut (θ_v) shall be taken as 36° , the value of k_v shall be determined from Clause 8.2.4.3 and the contribution of the fibres to the ultimate shear strength (V_{uf}) determined from Equation 16.4.4.2.1(1) with f'_w taken as equal to $f'_{1.5}$.

16.4.4.3 Minimum shear reinforcement

The minimum contribution from the total of the transverse steel reinforcement and fibres shall satisfy the following:

$$(V_{us} + V_{uf})_{\min} \geq 0.1b_v d_o \sqrt{f'_c} \text{ and} \quad \dots 16.4.4.3$$

$$\geq 0.6b_v d_o$$

16.4.5 Design with strut and tie models

Steel fibres may be used in combination with bar reinforcement to resist tension due to bursting in strut-and-tie models provided that—

- (a) at service loads the crack width is demonstrated to be not greater than 0.5 mm; and
- (b) at the strength limit state fibres do not contribute to more than 30% of the total calculated tension, with bar reinforcement supplied for the remainder.

NOTE: A higher level of crack control may be required for other limit state conditions, e.g. durability.

16.4.6 Fatigue

Steel fibres shall not be included in calculations for fatigue resistance unless demonstrated by testing.

16.4.7 Design for serviceability limit states

16.4.7.1 General

When an SFRC cross-section is uncracked, the full cross-section shall be assumed to be active and both concrete and steel assumed to be elastic in tension as well as in compression.

When an SFRC cross-section is cracked, the SFRC shall be assumed to be elastic in compression, and capable of sustaining a tensile stress equal to $1.1f'_{0.5}$.

NOTE: Where the value of $f'_{0.5}$ is not specified, it may be assumed that $f'_{0.5} = 1.1f'_{1.5}$ in Clause 16.4.7.1, Clause 16.4.7.4 and in Figure 16.4.7.3.2.

16.4.7.2 Stress limits

16.4.7.2.1 Concrete

The maximum compressive stress in the concrete at the serviceability limit states shall not exceed $0.6f_{cmi}(t)$. Under permanent effect loading, the maximum compressive stress in the concrete shall not exceed $0.4f_{cmi}(t)$.

NOTE: The satisfaction of limits on the concrete tensile stress at the serviceability limit state is not necessary if the member performance is satisfactory at the ultimate limit state.

16.4.7.2.2 Reinforcing steel

To avoid any inelastic deformation that could lead to large, permanently open cracks, tensile stresses in the reinforcement at the serviceability limit states shall not exceed $0.8f_{sy}$.

A2

16.4.7.3 Deflection control

16.4.7.3.1 General

The deflection of an SFRC member shall be calculated using the procedures outlined in Clauses 16.4.7.3.2 and 16.4.7.3.3. Allowance shall be made for the expected load history, the expected construction procedure and any anticipated deflections resulting from deformation of forms or settlement of props.

16.4.7.3.2 Short-term deflection

The short-term deflections due to external loads and prestressing, which occur immediately on their application, shall be calculated using the value of E_{cj} determined in accordance with Clause 3.1.2 and the value of the effective second moment of area of the member (I_{ef}). The value of I_{ef} shall be determined by rational calculation. Alternatively, I_{ef} may be determined from the values of I_{ef} at nominated cross-sections as follows:

- (a) For a simply supported span, the value at mid-span.
- (b) In a continuous beam—
 - (i) for an interior span, half the mid-span value plus one quarter of each support value; or
 - (ii) for an end span, half the mid-span value plus half the value at the continuous support.
- (c) For a cantilever, the value at the support.

For the purpose of the above determinations, the value of I_{ef} at each of the cross-sections nominated in Items (a) to (c) above shall be obtained from the instantaneous curvature $k_i = M_s^* / (E_{cj} I_{ef})$ calculated as the slope of the strain diagram in Figure 16.4.7.3.2(b) and obtained by satisfying the requirements for rotational and horizontal equilibrium of the stress distribution in Figure 16.4.7.3.2(c).

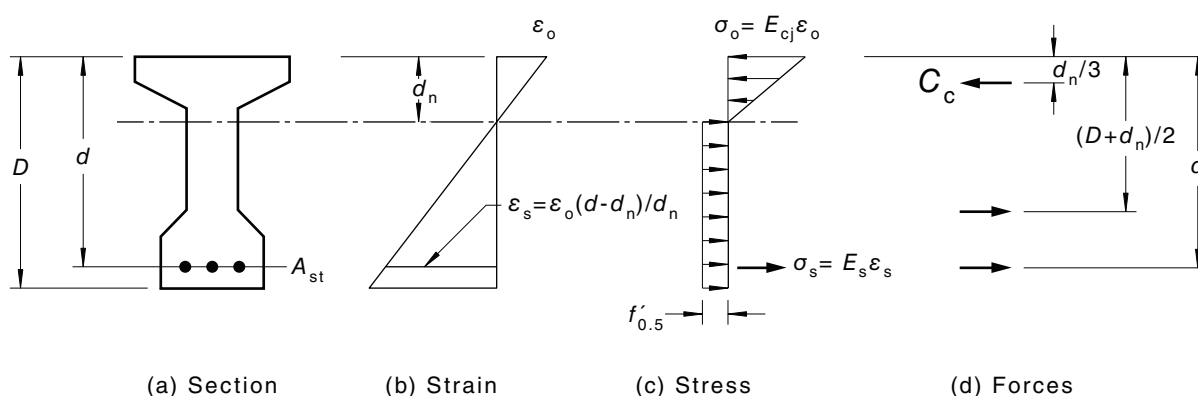


FIGURE 16.4.7.3.2 STRESS AND STRAIN DISTRIBUTION ON A CRACKED SECTION SUBJECTED TO APPLIED IN-SERVICE BENDING MOMENT (M_s^*)

16.4.7.3.3 Long-term deflection

For reinforced and prestressed SFRC flexural members, that part of the deflection that occurs after the short-term deflection shall be calculated as the sum of—

- (a) the shrinkage component of the long-term deflection, determined from the design shrinkage strain of concrete (ϵ_{cs}) (see Clause 3.1.7.1) and the principles of mechanics; and

- (b) the additional long-term creep deflections, determined from the design creep coefficient at any time (t) of concrete (ϕ_{cc}) (see Clause 3.1.8.3) and the principles of mechanics.

16.4.7.4 Flexural crack control

16.4.7.4.1 General

The calculated flexural crack width in a reinforced concrete beam or slab containing steel fibres in addition to conventional reinforcement shall be determined in accordance with Clause 8.6.2.3, except that the second term on the right hand side of Equation 8.6.2.3(2) shall be multiplied by k_{f1} and the maximum crack spacing $s_{r,max}$ determined using Equation 8.6.2.3(3) shall be multiplied by k_{f2} , where:

$$k_{f1} = \left(1 + \frac{f'_{1.5}}{f'_{ct}} \right) \quad \dots 16.4.7.4.1(1)$$

$$k_{f2} = \frac{(f'_{ct} - 1.1k_g f'_{1.5})}{f'_{ct}} \geq 0.25 \quad \dots 16.4.7.4.1(2)$$

NOTE: k_g introduced into SLS models.

16.5 DURABILITY

The minimum concrete grade and cover for SFRC in exposure classifications A, B1 and B2 shall be as for concrete without fibres and shall apply to the steel reinforcement only.

In exposure classification C1 or C2 and U, steel fibres, if present, shall be disregarded in calculations of strength limit states.

NOTES:

- 1 Steel fibres do not require concrete cover as specified for steel reinforcement in Clause 4.10.3.
- 2 SFRC may not be suitable in some exposure classification U environments.

16.6 FIRE

The structural performance of SFRC for fire shall be determined in accordance with Section 5 of the Standard.

16.7 PRODUCTION OF SFRC

16.7.1 Fibres

Steel fibres shall conform with ISO 13270:2013. The current 'Declaration of Performance' shall be supplied on request to the relevant authority.

NOTE: Steel fibres conforming with the EN 14889-1 'Fibres for concrete – Part 1: Steel fibres' conform with ISO 13270 and this clause.

16.7.2 Mixing of fibres

Fibres of the type and quantity specified shall be added in a controlled process ensuring that they are dispersed uniformly through the concrete mix. If added after the main mixing process, the concrete shall be remixed until the fibres have been completely dispersed throughout the batch.

NOTES:

- 1 Balling of fibres should be avoided.
- 2 A record of fibre content should be recorded for each batch.

16.7.3 Pre-construction testing of materials

Pre-construction testing in the form of initial type tests shall be undertaken in accordance with Table 16.7.3.

New initial type tests shall be undertaken when any of the following occur:

- (a) The concrete composition changes.
- (b) At least one source material of the concrete changes.
- (c) The results of compressive tests undertaken to AS 1379 do not meet the specification.
- (d) At least once in a period of 12 months.

The pre-construction tests shall be performed with the same materials and equipment that will be used during production.

TABLE 16.7.3
PRE-CONSTRUCTION TESTS

Material	Inspection/Test	Purpose
Steel fibres	Record and document delivery note	Verify that the delivery is in accordance with the order, and is shipped from the correct source
	Check and record ISO 13270 conformance	Verify that the fibres have the correct ISO or CE-label for use of steel fibres in concrete
Steel fibre content in the fresh concrete	Testing according to EN 14721 and Clause 16.7.5 on the basis of nine samples	Conformity with the target dosage. Verify homogeneous distribution of the steel fibres in the mix
Steel fibre concrete performance	Check limit of proportionality, and post-crack flexural strength in accordance to EN 14651 on a minimum of six beams	Verify that the performance is in accordance to the specification. The performance level serves as the reference for continuous production control

NOTE: The characteristic post-crack residual flexural strength shall be determined using the procedures given in Clause 16.3.3.7.

16.7.4 Factory production control

Factory production control in accordance with Table 16.7.4 shall be undertaken to establish a production process for the steel fibre reinforced concrete, and shall include the following:

- (a) Checking the correct concrete constituents are being used in production.
- (b) Steel fibres are checked against ISO or CE labelling.
- (c) Compressive tests undertaken to AS 1379.
- (d) Fibre content and distribution.

NOTE: The fibre type and dosage shall be shown on the delivery docket.

TABLE 16.7.4
ROUTINE PRODUCTION CONTROL

Material	Inspection/Test	Purpose	Frequency
Equipment inspection			
Automatic dosing equipment for steel fibres	Visual inspection	Assure correct functioning of dosing device	Once per production day
	Control of accuracy	Avoid improper fibre dosage	On installation Periodically In case of doubt
Materials inspection			
Steel fibres	Record and document delivery note	Verify that the delivery is in accordance with the order, and is shipped from the correct source	Each delivery
	Check and record ISO or CE label	Verify that the fibres have the correct ISO or CE label which matches the corresponding Declaration of Performance	Each delivery
	Visual control, measure fibre dimensions	Compare the fibre geometry with the fibres used for ITT	Each delivery
Production process inspection			
Fibre content in the fresh concrete	Testing according to EN 14721 and Clause 16.7.5	Conformity with the target dosage. Verify homogeneous distribution of the steel fibres in the mix	Beginning of each day and every /50 m ³ (manual dosing) /150 m ³ (auto dosing)
Concrete mix	Visual check	Correct mixing with correct fibre type and even fibre distribution without balling	Daily
Finished product inspection			
Fibre content-record	Record the quantity added	Check the content	Every batch
Steel fibre concrete performance	Check limit of proportionality, and post-crack flexural strength in accordance to EN 14651	Check performance level of the specification	At minimum of six beams over any 12 month period of production

16.7.5 Determining the steel fibre content

Steel fibre content shall be measured from samples taken from the production concrete and carried out in accordance with EN 14721.

In addition—

- (a) a sample shall be taken from the batch of concrete at unloading from the first third, middle third and final third of the batch;
- (b) each sample shall be a minimum of 7 litres;
- (c) the sample container shall be filled in one continuous pour and where possible directly from the discharge chute; and

- (d) wash-out, magnetic separation or validated automated dosage equipment shall be used.

NOTE: Automated dosing equipment refers to fibre distribution determined by electronic means through continuous measuring of the concrete as it's discharged from the concrete truck. The method needs to have been demonstrated to be validated for the designed usage and supply conditions and environment.

The steel fibre content shall be deemed to conform provided the criteria in Table 16.7.5 are met.

TABLE 16.7.5
CRITERIA OF ACCEPTANCE FOR STEEL FIBRE DOSAGE

Frequency	Test control	Criteria
Each sample	Each partial test	≥ 0.80 of the specified target dosage
Average of three samples from the batch	Each test	≥ 0.80 of the specified target dosage
Continuous control: average of >3 tests	Continuous control: average of >3 tests	≥ 0.90 of the specified target dosage

16.7.6 Sampling, testing and assessment for conformance of hardened SFRC

When concrete is specified by parameters other than strength grade, the method of production control and, if required, project control shall be specified together with the relevant conformance criteria.

Methods of control and assessment shall provide a reliable operating characteristic curve so that—

- (a) concrete with a proportion defective of 0.05 has a probability of acceptance of at least 50%; and
- (b) concrete with a proportion defective of 0.30 has a probability of rejection of at least 98%.

SECTION 17 MATERIAL AND CONSTRUCTION REQUIREMENTS

17.1 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR CONCRETE AND GROUT

17.1.1 Materials and limitations on constituents

Materials for concrete and grout, and limitations on their chemical content, shall conform with the relevant requirements of AS 1379.

17.1.2 Specification and manufacture of concrete

Concrete to which this Standard applies shall be—

- (a) specified as either normal-class or special-class and manufactured and supplied in accordance with AS 1379; and
- (b) handled, placed, compacted, finished and cured in accordance with this Standard, so that the hardened concrete will satisfy the design requirements for strength, serviceability, durability and other limit states.

Project assessment shall be specified for special-class concrete specified by strength grade as defined in AS 1379.

NOTE: Project assessment may be specified also for normal-class concrete and other special-class concrete as defined in AS 1379.

17.1.3 Handling, placing and compacting of concrete

Concrete shall be handled, placed and compacted so as to—

- (a) limit segregation or loss of materials;
- (b) limit premature stiffening;
- (c) produce a monolithic mass between planned joints or the extremities of members, or both;
- (d) completely fill the formwork to the specified level, expel entrapped air, and closely surround all reinforcement, tendons, ducts, anchorages, embedments and fixings; and
- (e) provide the specified finish to the formed surfaces of the member.

17.1.4 Finishing of unformed concrete surfaces

Unformed concrete surfaces shall be finished by methods which achieve the specified—

- (a) dimensions, falls, tolerances, or similar details relating to the shape and uniformity of the surfaces;
- (b) cover from the surfaces to reinforcement, tendons, ducts and embedments; and
- (c) texture of the surface.

17.1.5 Curing and protection of concrete

17.1.5.1 *Curing*

Concrete shall be cured continuously for a period of time so the design requirements for strength, serviceability and stripping are satisfied. To satisfy durability, curing requirements shall be not less than those given in Clauses 4.4 and 4.5.

Curing shall be achieved by the application of water to, accelerated curing of, or the retention of water in, the freshly cast concrete. Curing shall commence as soon as practicable after the finishing of any unformed surfaces has been completed. Where retention of water in the fresh concrete relies on the application to exposed surfaces of sprayed membrane-forming curing compounds, the compounds used shall conform with AS 3799.

Curing requirements for the various members of the structure shall be detailed in the project specification.

17.1.5.2 Protection

Freshly cast concrete shall be protected from the effects of rain, running water and freezing or drying prior to hardening. During the initial curing period the concrete shall be protected from freezing or drying.

17.1.6 Sampling and testing for conformance

17.1.6.1 General

Concrete for use in structures designed in accordance with this Standard, shall be assessed in accordance with AS 1379 for conformance with the specified parameters.

NOTE: When project assessment is required, the project specification should nominate responsibility for carrying out the relevant sampling, testing and assessment and, if these differ from or are not covered by AS 1379, should give details of how the assessment is to be made.

17.1.6.2 Concrete and grout specified by strength grade

Concrete and grout specified by strength grade shall satisfy the following criteria:

- (a) For each strength grade of concrete supplied to a project, the mean cylinder compressive strength (f_{cm}), as defined in AS 1379, shall be maintained within the limits specified in that Standard.
- (b) For concrete subject to project assessment—
 - (i) the slump of the supplied concrete shall be within the tolerance specified in AS 1379 for the relevant specified slump; and
 - (ii) in addition to Item (a), the mean compressive strength of the representative samples taken from the project shall be within the limits specified in AS 1379.
- (c) Conversion of cube strength to cylinder strength shall be determined by testing. In the absence of test data, cylinder strength may be taken as cube strength multiplied by 0.8.

NOTES:

- 1 'Strength grade' is defined in AS 1379 as 'the specified value of the characteristic compressive strength of the concrete at 28 days (f'_c)'.
- 2 The compressive strength of the concrete sampled, tested and assessed in accordance with AS 1379 indicates the potential strength of the supplied concrete, when placed, compacted and cured under optimum conditions; the responsibility of demonstrating rests on the supplier. The achievement of that potential on site is dependent upon the handling, placing, compacting and curing techniques actually used; the responsibility for which rests with the construction contractor (see Clauses 17.1.3 and 17.1.5). Information on appropriate site techniques may be found in HB 64 and HB 67.

17.1.6.3 Concrete specified by parameters other than strength grade

When concrete is specified by parameters other than strength grade, the method of production control and, if required, project control shall be specified together with the relevant conformance criteria.

The specified methods of control and assessment shall provide a reliable operating characteristic curve so that—

- (a) concrete with a proportion defective of 0.05 has a probability of acceptance of at least 50%; and
- (b) concrete with a proportion defective of 0.30 has a probability of rejection of at least 98%.

17.1.7 Rejection of concrete

17.1.7.1 Plastic concrete

Plastic concrete shall be liable to rejection if, after completion of mixing but prior to site handling—

- (a) the slump, determined in accordance with AS 1012.3.1, differs from the specified slump by more than the tolerances permitted in AS 1379;
- (b) the elapsed time from first introduction of the mixing water is outside the time interval allowed in AS 1379; or
- (c) the appearance and cohesiveness of a particular quantity is different from previously supplied quantities of the same specification.

17.1.7.2 Hardened concrete

Hardened concrete shall be liable to rejection if—

- (a) it does not satisfy the requirements of Clause 17.1.6;
- (b) it is porous, segregated, or honeycombed, or contains surface defects outside the specified limits; or
- (c) it fails to conform with the other requirements of this Standard.

17.1.7.3 Action on hardened concrete liable to rejection

Where hardened concrete is liable to rejection in terms of Clause 17.1.7.2, the concrete may be accepted if it can be demonstrated, either by calculation or by testing in accordance with Appendix B, that the structural adequacy and intended use of the affected members are not impaired. Otherwise, the concrete shall be rejected.

17.1.8 Requirements for grout and grouting

17.1.8.1 Grout properties

Grout shall be proportioned to give the desired properties for its designed use. Grout to be used in grouting prestressing ducts shall be able to be pumped through the duct, have low sedimentation and shrinkage, and contain no more than 750 mg of chloride ions per litre of grout.

17.1.8.2 Mixing and agitation

Grout shall be mixed in a mixer capable of producing a uniform grout of the specified fluidity and free from lumps of undispersed cement.

After mixing, grout shall be held in an agitation tank and kept in motion, to prevent settlement or segregation occurring, before it is pumped into its final position.

17.2 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR REINFORCING STEEL

17.2.1 Materials

17.2.1.1 Reinforcement

All reinforcement shall conform with AS/NZS 4671.

Main reinforcement shall be deformed Ductility Class N bars, or Ductility Class L or Ductility Class N welded wire mesh (plain or deformed). Fitments shall be deformed Ductility Class N bars, Ductility Class L wire or bar (plain or deformed), or plain Ductility Class N bar.

Ductility Class L reinforcement shall not be substituted for Ductility Class N reinforcement unless the structure is redesigned.

17.2.1.2 Protective coatings

A protective coating may be applied to reinforcement provided such coating does not reduce the properties of the reinforcement below those assumed in the design.

17.2.2 Fabrication

Reinforcement shall be fabricated in accordance with the following:

- (a) Reinforcement shall be fabricated to the shape and dimensions shown in the drawings and within the following tolerances:
 - (i) On any overall dimension for bars and mesh except where used as a fitment—
 - (A) for lengths up to 600 mm -25, +0 mm;
 - (B) for lengths over 600 mm -40, +0 mm.
 - (ii) On any overall dimension of bars or mesh used as a fitment—
 - (A) for deformed bars and mesh -15, +0 mm;
 - (B) for plain round bars and wire -10, +0 mm.
 - (iii) On the overall offset dimension of a cranked column bar -0, +10 mm.
 - (iv) For the sawn or machined end of a straight bar designed for use as an end-bearing splice, the angular deviation from square, measured in relation to the end 300 mm, shall be within 2°.
- (b) Bending of reinforcement shall conform with Clause 17.2.3.
- (c) Welding if required shall conform with AS/NZS 1554.3. Locational tack welding shall also conform with AS/NZS 1554.3.

17.2.3 Bending

17.2.3.1 General

Reinforcement shall be bent either—

- (a) cold, by the application of a force, around a pin of diameter conforming with Clause 17.2.3.2, so as to avoid impact loading of the bar and mechanical damage to the bar surface; or
- (b) hot, provided—
 - (i) the steel is heated uniformly through and beyond the portion to be bent;
 - (ii) the temperature of the steel does not exceed 600°C;
 - (iii) the bar is not cooled by quenching; and
 - (iv) if during heating the temperature of the bar exceeds 450°C, the design yield strength of the steel after bending is taken as 250 MPa and the design checked for strength.

Reinforcement that has been bent and subsequently straightened or bent in the reverse direction shall not be bent again within 20 bar diameters of the previous bend.

Bars shall not be bent using impact, such as with hammers.

Reinforcement partially embedded in concrete may be field-bent provided the bending conforms with Items (a) or (b) above and the bond of the embedded portion is not impaired thereby.

17.2.3.2 *Site rebending*

Where site rebending is required, the following procedures shall be followed:

- (a) Rebending of bars shall be undertaken using a rebending tool. Bars shall be rebent against a flat surface or a pin with a diameter that is at least equal to or greater than the minimum pin size as specified in Clause 17.2.3.3. Bars shall not be over-bent beyond the original bend, typically 90°.
- (b) If scabbling tools are to be used near bars because of concrete leakage or contamination, extreme care shall be exercised to prevent any impact or damage to the bars.
- (c) The bar shall be positioned with the initial bend of the bar clear of the concrete.
- (d) A bar shall only be rebent once at the same location.

17.2.3.3 *Internal diameter of bends or hooks*

The nominal internal diameter of a reinforcement bend or hook shall be taken as the external diameter of the pin around which the reinforcement is bent. The diameter of the pin shall be not less than the value determined from the following:

- (a) For fitments of—
 - (i) 500L bars $3d_b$;
 - (ii) R250N bars $3d_b$; and
 - (iii) D500N bars $4d_b$.
- (b) For reinforcement, other than that specified in Item (c) and Item (d) of any grade $5d_b$.
- (c) For reinforcement, in which the bend is designed to be subsequently straightened or rebent, of—
 - (i) 16 mm diameter or less $4d_b$;
 - (ii) 20 mm diameter or 24 mm $5d_b$; and
 - (iii) 28 mm diameter or greater $6d_b$.

Any such straightening or rebending shall be clearly specified or shown in the drawings.
- (d) For reinforcement that is epoxy-coated or galvanized, either before or after bending, of—
 - (i) 16 mm diameter or less $5d_b$;
 - (ii) 20 mm diameter or greater $8d_b$.

17.2.4 **Surface condition**

At the time concrete is placed, the surface condition of reinforcement shall be such as not to impair its bond to the concrete or its performance in the member. The presence of millscale or surface rust shall not be cause for rejection of reinforcement under this clause.

17.2.5 **Fixing**

All reinforcement, including secondary reinforcement provided for the purpose of maintaining main reinforcement and tendons in position, shall be supported and maintained in position within the tolerances given in Clause 17.5.3 until the concrete has hardened. Bar

chairs and spacers used for this purpose shall conform with AS/NZS 2425 and be specified with a strength grade and spacing, to support the mass of reinforcement and applied construction loads. Tie wire shall be used to maintain position unless the bar chair has an in-built clip. Tie wire shall not be finished within the concrete cover.

NOTE: Off-cuts of tie wire should be removed before placement of concrete.

17.2.6 Lightning protection by reinforcement

Where lightning protection is to be provided by the reinforcement, the reinforcement shall conform with the relevant requirements of AS/NZS 1768.

17.3 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR PRESTRESSING DUCTS, ANCHORAGES AND TENDONS

17.3.1 Materials for ducts, anchorages and tendons

17.3.1.1 Ducts

Sheaths and removable formers used to form ducts shall be capable of maintaining their required cross-section and profile during construction.

17.3.1.2 Anchorages

The quality and properties of anchorages shall be established by testing.

17.3.1.3 Tendons

Prestressing tendons shall conform with AS/NZS 4672.1, as applicable.

Tendons shall not be galvanized.

Hard-drawn, high tensile steel wire, which has not been stress-relieved, shall not be used for wire winding unless its elongation, tested in accordance with AS/NZS 4672.1, is 3.5% or greater.

Plain wire shall not be used for pretensioning.

17.3.2 Construction requirements for ducts

17.3.2.1 Surface condition

When concrete is placed, the outside surface of sheaths and formers for ducts shall be such as not to impair bond of the concrete to the duct. Immediately before grouting, the inside surfaces of sheaths shall be such as not to impair bond of the grout to the duct.

The technique for withdrawal of an extractable core shall not damage the formed duct.

17.3.2.2 Sealing

Prior to the placing of concrete, ducts shall be sealed at the ends and at all joints, to exclude concrete, or other matter.

17.3.2.3 Fixing

Ducts shall be supported and fixed at regular intervals so the required tendon profile will be maintained in accordance with Clause 17.5.3.

17.3.3 Construction requirements for anchorages

17.3.3.1 Fixing

Anchorage shall be fixed strictly in accordance with the supplier's recommendations and the following:

- (a) The anchorage shall be square to the line of the tendon.
- (b) The duct shall be securely attached to the anchorage so it provides a grout-tight joint between the duct and the anchorage.

- (c) Where the anchorage is fixed to the formwork, the joint between the two parts shall be grout-tight.

17.3.3.2 Surface condition

At the time concrete is placed, the surface condition of the anchorage shall be such as not to impair its bond to the concrete.

17.3.4 Construction requirements for tendons

17.3.4.1 Fabrication

Tendons shall be fabricated in accordance with the following:

- (a) Cutting of tendons shall be carried out so damage to tendons, ducts and anchorages is avoided.
- (b) Tendons shall not be welded.
- (c) Prestressing bars shall be within manufacturing tolerances and not bent in the threaded portion.

Small adjustments on site shall be carried out cold. If the bar temperature is lower than 10°C, the bar temperature shall be raised above this value by means of steam or hot water.

17.3.4.2 Protection

Before stressing, tendons shall be protected from stray current arcing and splashes from the cutting operation of an oxy-acetylene torch or an arc-welding process.

The threaded ends of prestressing bars shall be protected at all times.

If tendons are to have a coating or wrapping, such coating or wrapping shall be inert with respect to both the steel and the concrete.

After stressing and anchoring, all tendons and anchorages shall be protected from physical damage and corrosion.

17.3.4.3 Surface condition

The surface condition of tendons shall be such as not to impair bond to the concrete or grout, or performance in the member.

The presence of surface rust shall not be cause for rejection of ducts under this clause unless the steel is visibly pitted.

17.3.4.4 Fixing

All tendons shall be supported and maintained in position within the permissible tolerances given in Clause 17.5.3 until the concrete has hardened.

17.3.4.5 Tensioning

Tensioning of tendons shall be carried out in a safe manner and in accordance with the following:

- (a) The stressing procedure shall ensure the force in a tendon increases at a uniform time rate and that the force is transferred gradually to the concrete.
- (b) The prestressing force applied to the tendon shall be measured at the jack by measuring the jack pressure. The prestressing force shall be measured to an accuracy of $\pm 3\%$.
- (c) The tendon extension shall be measured.
- (d) A check shall be made for each tendon, on the correlation between the measured extension and the calculated extension derived from the prestressing force, using the load-elongation curves for the tendons and assumed friction values for the cable. Any disparity between the two figures greater than 10% of the calculated extension shall be investigated.

- (e) No stressing shall be carried out when the temperature of the surrounding air is lower than 0°C.

17.3.4.6 *Maximum jacking forces*

The maximum force to be applied to a tendon during the stressing operation shall not exceed—

- (a) for pretensioned tendons $0.80f_{pb}A_p$;
 (b) for stress-relieved post-tensioned tendons $0.85f_{pb}A_p$; or
 (c) for post-tensioned tendons and bars not stress-relieved $0.75f_{pb}A_p$.

17.3.4.7 *Grouting*

Ducts containing post-tensioned tendons shall be completely filled with grout, conforming with Clause 17.1.8, as soon as practicable after stressing. Grouting shall not be carried out when the temperature of the surrounding air is lower than 5°C.

Precautions shall be taken to prevent corrosion for the tendons if the elapsed period prior to grouting is likely to exceed four weeks.

17.3.5 **Construction requirements for unbonded tendons**

Unbonded tendons shall not be permitted except in slabs on the ground. Where so used, the requirements of Clauses 17.3.4.1 to 17.3.4.6 shall apply, and the tendons shall be protected against corrosion.

17.4 **CONSTRUCTION REQUIREMENTS FOR JOINTS AND EMBEDDED ITEMS**

17.4.1 **Location of construction joints**

Construction joints shall be located in accordance with the following:

- A1 | (a) Construction joints designed in accordance with Clause 19.1.2 shall be located to facilitate the placement of concrete in accordance with Clause 17.1.3.
- A1 | (b) Where an interruption to the placing of concrete occurs such that the requirements of Clause 17.1.3(c) or Clause 17.1.3(d) or Clause 17.1.3(e) cannot be fulfilled, a construction joint conforming with Clause 19.1.2 shall be made at an appropriate location.

17.4.2 **Embedded and other items not shown in the drawings**

- A1 | Where an embedded item, driven fixing device or hole is required, it shall be located so that the behaviour of the members is not impaired (see Clause 19.2).

17.5 **TOLERANCES FOR STRUCTURES AND MEMBERS**

17.5.1 **General**

For the purposes of the strength requirements of this Standard, the position of any point on the surface of a concrete member shall conform with Clause 17.5.2.

NOTE: More stringent tolerances may be required for reasons of serviceability, fit of components, or aesthetics of the structure.

For formed surfaces, the tolerances given in AS 3610 take precedence, unless those in Clause 17.5.2 are more stringent. For unformed plane surfaces, the flatness tolerances and the methods for measuring them shall be detailed in the project specification, and shall be not greater than the relevant values given in Clause 17.5.2.

17.5.2 Tolerances for position and size of structures and members

17.5.2.1 *Absolute position*

The deviation from the specified position shall not exceed the following:

- (a) In plan, for a point on the surface of a column or wall at any floor level—
 - (i) in the first 20 storeys of any building.....40 mm horizontally; and
 - (ii) for subsequent storeys, an increase of 15 mm horizontally for each additional 10 storeys or part thereof.
- (b) In elevation, for a point on the top surface of a floor or the soffit of a beam or slab adjacent to a column or wall 40 mm vertically.

17.5.2.2 *Floor-to-floor plumb*

In any column or wall, the deviation from plumb, measured floor-to-floor, shall not exceed 1/200 times the dimension between the floors or 10 mm, whichever is the greater.

17.5.2.3 *Deviation from specified dimensions*

The deviation from any specified height, plan, or cross-sectional dimension, shall not exceed 1/200 times the specified dimension or 5 mm, whichever is the greater.

17.5.2.4 *Deviation from surface alignment*

The deviation of any point on a surface of a member, from a straight line joining any two points on the surface, shall not exceed 1/250 times the length of the line.

17.5.3 Tolerance on position of reinforcement and tendons

The deviation from the specified position of reinforcement and tendons shall not exceed the following:

- (a) For positions controlled by cover—
 - (i) in beams, slabs, columns and walls–5, +10 mm;
 - (ii) in slabs-on-ground..... –10, +20 mm; and
 - (iii) in footings cast in the ground..... –10, +40 mm,
 where a positive value indicates the amount the cover increases and a negative value indicates the amount the cover decreases.
- (b) For positions not controlled by cover, namely—
 - (i) the location of tendons on a profile..... 5 mm;
 - (ii) the position of the ends of reinforcement 50 mm; and
 - (iii) the spacing of bars in walls and slabs and of fitments in beams and columns..... 10% of the specified spacing or 15 mm, whichever is greater.

17.6 FORMWORK

17.6.1 General

The materials, design and construction of formwork shall conform with AS 3610. Stripping of forms and removal of formwork supports from members cast in situ shall conform with the requirements of Clause 17.6.2 where these are more stringent than the relevant requirements of AS 3610.

17.6.2 Stripping of forms and removal of formwork supports

17.6.2.1 *General*

The stripping of forms and the removal of formwork supports shall conform with the following:

- (a) Forms shall not be stripped or any formwork supports removed until the part of the member that will be left unsupported has attained strength determined in accordance with Section 2 to support, with safety and without detriment to its designed use, its own weight and any superimposed loads due to concurrent or subsequent construction works.
- (b) Removal of formwork supports shall be carried out in a planned sequence so the concrete structure will not be subject to any unnecessary deformation, impact, or eccentric loading during the process.
- (c) Removal of formwork from vertical surfaces shall be carried out in accordance with Clause 17.6.2.2.
- (d) Stripping of forms, from the soffits of reinforced slabs and beams between formwork supports, shall be carried out in accordance with Clause 17.6.2.3 or Clause 17.6.2.4. Where backpropping is used, the procedure shall conform with the requirements of AS 3610.
- (e) Removal of formwork supports from the soffits of reinforced slabs or beams shall be carried out in accordance with—
 - (i) Clause 17.6.2.5 for members not supporting structures above; or
 - (ii) Clause 17.6.2.6 for multistorey structures.
- (f) Stripping of forms and removal of formwork supports from prestressed beams and slabs shall be carried out in accordance with Clause 17.6.2.7.
- (g) Where formwork is stripped before the end of the specified curing period for the concrete element, exposed surfaces shall be cured until at least the end of the specified curing period.

17.6.2.2 *Removal of formwork from vertical surfaces*

Formwork shall not be removed from vertical surfaces unless the concrete in the member has achieved strength determined in Clause 3.1.1.2 to withstand potential damage to its surfaces.

When formwork is stripped at less than 18 hours after casting, extra care shall be exercised to avoid surface damage during stripping.

17.6.2.3 *Stripping of soffit forms from reinforced beams and slabs where control samples are available*

Where control samples have been taken, cured and tested in accordance with Clause 17.6.2.8, soffit forms may be stripped from between the formwork supports of reinforced beams and slabs if—

- (a) the elapsed time between casting of the concrete and the commencement of stripping is greater than three days; and
- (b) the spans between the remaining formwork supports are such that the member will remain uncracked under the action effects of bending and shear due to the maximum concurrent or subsequent construction loads.

In determining whether sufficient curing time has elapsed, the design resistance of the member shall be taken as ϕR_u (see Clause 2.2), and the mean in situ strength of the concrete is determined from the average strength of the control samples in accordance with Clause 3.1.1.2.

17.6.2.4 *Stripping of soffit forms from reinforced slabs of normal-class concrete*

For reinforced slabs of normal-class concrete, for which an early-age strength has been specified and which are continuous over formwork supports, the period of time between casting of the concrete and the commencement of stripping of the forms between formwork

supports shall be not less than that given in Table 17.6.2.4 for the average ambient temperature over the period. The periods given in the table shall be increased if—

(a) $L_s / D > 280 / \sqrt{(D + 100)}$. . . 17.6.2.4

where

L_s is the span between the formwork supports; and

D is the overall depth of the concrete member.

- (b) the superimposed construction load is greater than 2.0 kPa; or
- (c) the average ambient temperature over the period is less than 5°C, in which case the periods shall be increased by half a day for each day the daily average temperature was between 2°C and 5°C, or by a whole day for each day the daily average temperature was below 2°C.

TABLE 17.6.2.4
STRIPPING OF FORMWORK FROM REINFORCED
SLABS CONTINUOUS OVER FORMWORK SUPPORTS

Average ambient temperature over the period (T) °C	Period of time before stripping <i>normal-class</i> concrete with specified early-age strength Days
$T > 20$	4
$20 \geq T > 12$	6
$12 \geq T > 5$	8

17.6.2.5 *Removal of formwork supports from reinforced members not supporting structures above*

A2 | For the purpose of determining the minimum period before any undisturbed supports or backdrops can be removed from the soffits of reinforced members not supporting a structure above, it may be taken that the requirements of Clause 17.6.2.1(a) shall be deemed to be satisfied if either—

- (a) it can be demonstrated by calculations, based on known or specified early-age strengths that, at the time of removal, the concrete has gained sufficient strength so that the degree of cracking or deformation that will occur, then or subsequently, is not greater than that which would occur if the design serviceability load were applied to the member when the concrete has attained its required design strength, determined by Clause 2.3; or
- (b) in the absence of any early-age strength data, the period of time is not less than that given in Table 17.6.2.5 for the average ambient temperature over the period.

The periods given in Table 17.6.2.5 shall be increased if—

- (i) the superimposed construction load is greater than 2.0 kPa; or
- (ii) the average ambient temperature is less than 5°C, in which case the periods shall be increased by half a day for each day the daily average temperature was between 2°C and 5°C, or by a whole day for each day the daily average temperature was below 2°C.

TABLE 17.6.2.5
REMOVAL OF FORMWORK SUPPORTS FROM SLABS
AND BEAMS NOT SUPPORTING STRUCTURES ABOVE

Average ambient temperature over the period (T) °C	Period of time before removal of all formwork supports from reinforced members Days
$T > 20$	12
$20 \geq T > 12$	18
$12 \geq T > 5$	24

17.6.2.6 *Removal of formwork supports from reinforced members in multistorey structures*

In multistorey structures, the number of storeys (including the lowest storey) that are to remain supported by formwork at any one time and the maximum spacing of the formwork supports in any storey, shall be calculated on the basis of the relevant properties of the concrete in each floor at that time and the interaction between the formwork supports and the concrete structure.

Where removal of formwork supports from a storey will result in the floors above being supported mainly by formwork and supported concrete construction, all supported and supporting floors and beams shall be checked by calculation for cracking and deflection under the resulting loads. Removal of formwork supports from that storey shall be permitted only if the magnitude of the cracks and deflections so calculated will not impair the strength or serviceability of the completed structure.

No undisturbed supports or backprops shall be removed within two days of the placing of any slab directly or indirectly supported by such supports.

17.6.2.7 *Stripping of forms and removal of supports from soffits of prestressed concrete slabs and beams*

Formwork shall not be stripped and formwork supports not removed from the soffits of prestressed concrete slabs or beams until the strength of the concrete in the member and the number of tendons stressed are such as to provide the necessary strength to carry the permanent action (dead load) and imposed actions due to construction loads, and meet the associated serviceability and other limit state requirements.

17.6.2.8 *Control tests*

If specified, control test-samples of the concrete shall be taken where it is intended that removal of formwork or the stressing of tendons will occur before the concrete has attained the strength assumed in the design of the member.

Control test-samples shall be taken at a minimum frequency of one sample for each 50 m³, or part thereof, of a concrete grade placed on any one day and the sample specimens stored and cured under conditions similar to those of the concrete in the work.

At least two specimens from each grade shall be tested for strength at the desired time of stripping or stressing and the strength of the concrete at that age assessed on the basis of the average strength of the specimens.

17.7 PREFABRICATED CONCRETE STRUCTURES

17.7.1 General

Except where the provisions of Clause 17.7.4 govern, the minimum provisions of Clause 17.7.1 through to Clause 17.7.3 for structural integrity shall apply to all prefabricated concrete structures.

A2 | 17.7.2 Prefabricated elements as floor or roof diaphragms

Connections between diaphragms and those members being laterally supported shall have a designed tensile strength capable of resisting not less than 4.5 kN per linear metre.

17.7.3 Vertical integrity ties

All vertical structural members except for non-loadbearing elements shall have connections at horizontal joints in accordance with the following requirements:

- (a) Connections between prefabricated columns shall have a designed strength in tension not less than $f_{vt} \times A_g$, where f_{vt} is taken as 1.4 MPa and A_g is in mm². For columns with a larger cross sections than required for strength, a reduced effective area A_g may be used based on the cross section required but shall be not less than one-half the total area.
- (b) Prefabricated wall panels shall have a minimum of two ties per panel, with a designed strength not less than 45 kN per tie.
and
- (c) When design forces result in no tension at the base, the ties required by Clause 17.7.3(b) shall be anchored into a reinforced concrete floor slab-on-ground or footing.

Connection details that rely solely on friction caused by gravity loads shall not be permitted.

17.7.4 Prefabricated concrete loadbearing walls

For prefabricated concrete loadbearing wall structures ties between structural elements shall be provided to meet the requirements of the loadbearing wall system and designed using principles of mechanics.

17.7.5 Grouted ducts to joints

Grouted ducts used to join prefabricated elements shall be fully confined using Ductility Class N reinforcement anchored perpendicular to the duct in accordance with Section 11.

17.7.6 Seating of prefabricated floor elements

Seating of prefabricated floor elements shall have seating at least 1.5 times the Ultimate Limit State drifts.

SECTION 18 DESIGN FOR FATIGUE

18.1 GENERAL

The fatigue design action Q_{fat} and the foreseen number of stress cycles n_{sc} shall be determined in accordance with Clause 2.1.5. The load combinations shall be determined in accordance with Clause 2.5.2.3.

NOTE: Fatigue is a strength limit state considered at the operational/serviceability load combination.

In the case of prestressed members, the relevant section shall be assessed for sensitivity to cracking. If any load combinations cause tensile stresses at the concrete surface, the stress ranges for concrete, reinforcing steel and tendons shall be calculated assuming a cracked section.

Nominal stresses shall be calculated at the site of potential fatigue initiation. The critical section for fatigue analysis shall be taken in accordance with Section 6, including Clause 6.2.1 and Clause 6.2.3. Additional checks shall be undertaken at changes in section or changes in quantity or direction of reinforcement.

NOTE: A support is any column, wall or supporting beam.

The concrete compression stress under the permanent design actions $[G, \gamma_p P]$ shall not exceed $0.45 f'_c$.

18.2 MAXIMUM COMPRESSIVE STRESS IN CONCRETE

Detailed fatigue design for concrete in compression is not required if the maximum calculated stresses $\sigma_{c,\text{max}}$ under the combination of loads specified in Clause 2.5.2.3 satisfy the following:

$$\eta_c \sigma_{c,\text{max}} \leq 0.45 \phi_{c,\text{fat}} f_{c,\text{fat}} \quad \dots 18.2(1)$$

where

$\sigma_{c,\text{max}}$ = maximum compressive stress at the extreme fibres under consideration, compression is taken as positive: $[\{G, P, \Psi_s Q\}, Q_{\text{fat}}]$

$$f_{c,\text{fat}} = 0.85 \beta_{cc}(t_o) f'_c \left(1 - \frac{f'_c}{400}\right)$$

$$\beta_{cc}(t_o) = e^{\left[s(1 - \sqrt{28/t_o})\right]}$$

s = coefficient of strength gain given in Table 18.3

η_c = concrete stress gradient factor and may be taken as equal to 1.0 or determined as follows:

$$\eta_c = \frac{1}{1.5 - 0.5(\sigma_{c,300} / \sigma_{c,\text{max}})} \quad \text{but not taken as less than 0.67}$$

$\sigma_{c,300}$ = compressive stress at a depth of 300 mm from the extreme compressive fibre for same load case as $\sigma_{c,\text{max}}$, but not less than zero (compression is positive)

t_o = age of concrete at time of first cyclic loading in days.

TABLE 18.2
COEFFICIENT OF STRENGTH GAIN $\beta_{cc}(t_0)$

Concrete	s	t_0			
		7d	56d	90d	360d
High early strength	0.20	0.82	1.06	1.09	1.16
Normal	0.25	0.78	1.08	1.12	1.20
Delayed strength	0.38	0.68	1.12	1.18	1.32

The fatigue requirements under the load combination specified in Clause 2.5.2.3 shall be deemed to be satisfied if the foreseen number of effective stress cycles (n_{sc}) is less than or equal to the number of resisting stress cycles to failure (N)—

$$\log(n_{sc}) \leq \log(N) \quad \dots 18.2(2)$$

The number of resisting stress cycles are calculated from—

$$\log(N_1) = \frac{8}{Y-1}(S_{cd,max} - 1) \quad \dots 18.2(3)$$

$$\log(N_2) = 8 + \frac{8 \ln(10)}{Y-1}(Y - S_{cd,min}) \log\left(\frac{S_{cd,max} - S_{cd,min}}{Y - S_{cd,min}}\right) \text{ and} \quad \dots 18.2(4)$$

with

$$Y = \frac{0.45 + 1.8 S_{cd,min}}{1 + (1.8 - 0.3 S_{cd,min}) S_{cd,min}} \quad \dots 18.2(5)$$

where—

- (a) if $\log(N_1) \leq 8$, then $\log(N) = \log(N_1)$;
- (b) if $\log(N_1) > 8$, then $\log(N) = \log(N_2)$;

and

$\sigma_{c,max}$ = maximum compressive stress at the extreme fibre under consideration, compression measured positive, $\gamma_F [\{G, P, \psi_s Q\}, Q_{fat}]$

$S_{cd,max}$ = maximum compressive stress level calculated as $\frac{\eta_c \sigma_{c,max}}{\phi_{c,fat} f_{c,fat}}$

$\sigma_{c,min}$ = minimum compressive stress at the extreme fibre under consideration, taken as zero if tensile, $\gamma_F [\{G, P, \psi_s Q\}, Q_{fat}]$

$S_{cd,min}$ = minimum compressive stress level calculated as $\frac{\eta_c \sigma_{c,min}}{\phi_{c,fat} f_{c,fat}}$ but not greater than 0.8

For multiple cycles with variable amplitudes, the fatigue damage may be determined using—

$$D_{Ed} = \sum_{i=1}^m \frac{n_i}{N_i} \leq 1.0 \quad \dots 18.2(3)$$

where

m = number of intervals with constant amplitude

n_i = foreseen number of constant amplitude cycles in interval i

N_i = ultimate number of constant amplitude cycles in interval i that can be carried before failure

18.3 PLAIN CONCRETE WITH COMPRESSION-TENSION STRESS

For plain (unreinforced) concrete in compression with tension stress, the maximum tensile stress in the concrete shall be limited by—

$$\begin{array}{l|l} \text{A1,} & \sigma_{\text{ct,max}} \leq \sigma_{\text{c,max}} / 38.5 \quad \dots 18.3(1) \\ \text{A2} & \end{array}$$

$$\log(N) = 9 \left(1 - \frac{\sigma_{\text{c,max}}}{\phi_{\text{c,fat}} f_{\text{c,fat}}} \right) \quad \dots 18.3(2)$$

where

$\sigma_{\text{ct,max}}$ = maximum tensile stress

$\sigma_{\text{c,max}}$ = maximum compressive stress

$$f_{\text{c,fat}} = 0.85 \beta_{\text{cc}} (t_o) f'_c \left(1 - \frac{f'_c}{400} \right)$$

$$\text{NOTE: } \sigma_{\text{c,max}} \leq \phi_{\text{c,fat}} f_{\text{c,fat}} \left(1 - \frac{\log(n_{\text{sc}})}{9} \right).$$

18.4 PLAIN CONCRETE WITH PURE TENSION OR COMBINED TENSION-COMPRESSION STRESS

For plain (unreinforced) concrete with pure tension or in combined tension-compression, the maximum tensile stress shall be limited by—

$$\begin{array}{l|l} \text{A1} & \sigma_{\text{ct,max}} > \sigma_{\text{c,max}} / 38.5 \quad \dots 18.4(1) \\ \text{A2} & \end{array}$$

$$\log(N) = 12 \left(1 - \frac{\sigma_{\text{ct,max}}}{\phi_{\text{c,fat}} f'_{\text{ct}}} \right) \quad \dots 18.4(2)$$

where

$\sigma_{\text{ct,max}}$ = maximum tension stress

$\sigma_{\text{c,max}}$ = maximum compression stress

$$\begin{array}{l|l} \text{A2} & \text{NOTE: } \sigma_{\text{ct,max}} \leq \phi_{\text{c,fat}} f'_{\text{ct}} \left(1 - \frac{\log(n_{\text{sc}})}{12} \right). \end{array}$$

18.5 SHEAR LIMITED BY WEB COMPRESSIVE STRESSES

The maximum shear force in flexural members under the combination of permanent effects and the fatigue design loading shall not exceed $0.6\phi V_{\text{u,max}}$, where $V_{\text{u,max}}$ is calculated in accordance with Clause 8.2.3.3.

18.6 SHEAR IN SLABS

18.6.1 General

The maximum shear force in concrete slabs, as determined in accordance with Clause 9.3 under the combination of permanent effects and the fatigue design loading, shall be limited to the values specified in this clause.

18.6.2 Allowable shear force under cyclic loading

The calculated shear force under fatigue design loads shall not exceed:

- (a) For slabs where the number of stress cycles is not greater than 2×10^6 and where the slab can act as a wide beam and a shear failure could occur across a substantial width, $0.6\phi V_u$; otherwise $0.54\phi V_u$.

If the longitudinal tensile reinforcement ratio $(A_{st} + A_{pt})/(bd_o)$ is less than 0.01, the permissible shear shall be further reduced by multiplying the permissible value by the factor $(100(A_{st} + A_{pt})/(bd_o))^{1/3}$.

- (b) Where the potential failure surface could form a truncated cone or pyramid around a support or loaded area, the maximum calculated shear shall not exceed $0.5\phi V_u$ specified in Clause 9.3.4.

18.7 ADJUSTMENT FACTOR FOR BOND BEHAVIOUR IN REINFORCING AND PRESTRESSING STEEL

To account for the difference in bond behaviour between reinforcing and prestressing steel, the tensile stress range in reinforcing steel shall be multiplied by the factor η_s , unless a more refined method is used:

$$\eta_s = \frac{1 + (A_p / A_s)}{1 + (A_p / A_s) \sqrt{\xi (d_b / d_p)}} \quad \dots 18.7$$

where

η_s = a factor which increases the stress in the reinforcing steel due to differences in bond behaviour between prestressing and reinforcing steel

A_s = area of reinforcing steel

A_p = area of prestressing steel

d_b = smallest diameter of the reinforcing steel in the relevant cross-section

d_p = diameter of the prestressing steel (for bundles an equivalent diameter $1.6\sqrt{A_p}$ is chosen, where A_p is the cross-section area of the bundle)

For post-tensioned members:

ξ = 0.2 for smooth prestressing steel

ξ = 0.4 for strands

ξ = 0.6 for ribbed prestressing wires

ξ = 1.0 for ribbed prestressing bars

For pretensioned members:

ξ = 0.6 for strands

ξ = 0.8 for ribbed prestressing steels

18.8 TENSILE STRESS RANGE IN STEEL

The fatigue strength for nominal stress range is given by a series of $\log(\Delta\sigma_R) - \log(N)$ curves ($S-N$ curves) that correspond to the reinforcement detail categories.

Detailed fatigue design for reinforcement is not required if the maximum calculated stress range $\Delta\sigma_s$ under the applied design action from AS 1170.0 with load combination specified in Clause 2.5.2.3, satisfy the following criteria:

Unwelded straight reinforcement $\Delta\sigma_s \leq 70 \text{ MPa}$

Welded or bent reinforcement $\Delta\sigma_s \leq 35 \text{ MPa}$

The damage of a single stress amplitude $\Delta\sigma_s$ shall be determined by using the corresponding $S-N$ curves given in Figure 18.8 for reinforcing and prestressing steel. The applied design action from AS 1170.0 shall be factored in accordance with Clause 2.5.2.3.

The fatigue requirement will be met if the calculated maximum acting stress range, $\Delta\sigma_s$, satisfies the condition—

$$\Delta\sigma_s \leq \phi_{s,\text{fat}} \Delta\sigma_{\text{Rsk}}(n_{\text{sc}}) \quad \dots 18.8(1)$$

where

$\Delta\sigma_s$ = maximum nominal steel stress range under the acting load combination

$\phi_{s,\text{fat}}$ = fatigue strength reduction factor for steel, taken as 0.85

$\Delta\sigma_{\text{Rsk}}(n_{\text{sc}})$ = resisting stress range relevant to n_{sc} cycles obtained from a characteristic fatigue strength function

$$\Delta\sigma_{\text{Rsk}}(n_{\text{sc}}) = \Delta\sigma_{\text{Rsk}}(N_{\text{Rsk}}) \left(\frac{N_{\text{Rsk}}}{n_{\text{sc}}} \right)^{1/m}$$

N_{Rsk} = number of cycles for resistance of constant amplitude stress $\Delta\sigma_{\text{Rsk}}$

$m = m_1$ for $n_{\text{sc}} \leq N_{\text{Rsk}}$; or

$m = m_2$ for $n_{\text{sc}} > N_{\text{Rsk}}$

Values of parameters N_{Rsk} , m_1 , m_2 and $\Delta\sigma_{\text{Rsk}}$ are given in Table 18.8.

A2

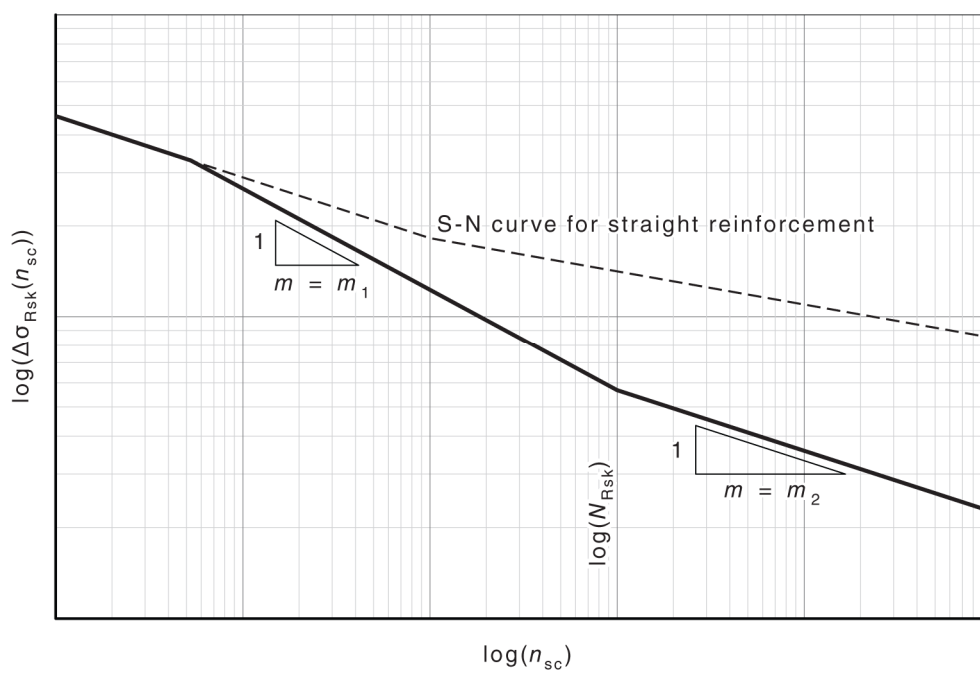


FIGURE 18.8 INDICATIVE S-N CURVE FOR REINFORCING STEEL AND TENDONS

TABLE 18.8
PARAMETERS FOR CHARACTERISTIC S-N CURVES FOR REINFORCING
STEEL AND TENDONS

Category	Reinforcement or tendon detail	N_{Rsk}	m_1	m_2	$\Delta\sigma_{Rsk}(N_{Rsk})$
	Reinforcement				
A	Straight reinforcement $d_b \leq 16$	10^6	5	9	210
B	Straight reinforcement $d_b > 16$	10^6	5	9	$(240 - 2d_b)$
C	Bent reinforcement $d_b \leq 16$ (see Note 2)	10^6	5	9	$210 k_d$
D	Bent reinforcement $d_b > 16$ (see Note 2)	10^6	5	9	$(240 - 2d_b)k_d$
E	Welded bar and wire mesh (see Note 1)	10^7	3	5	50
F	Mechanical connectors	10^7	3	5	50
G	Marine environment (see Note 3)	10^7	3	5	65
	Tendons				
H	Pre-tensioning	10^6	5	9	185
I	Post-tensioning—Single strands in plastic ducts	10^6	5	9	185
J	Post-tensioning—Straight tendons or curved tendons in plastic ducts	10^6	5	10	150
K	Post-tensioning—Curved tendons in steel ducts	10^6	5	7	120
L	Post-tensioning—Splicing devices	10^6	5	5	80

Definitions:

$$k_d = 0.35 + 0.026(d_i/d_b) \leq 1.0$$

d_i = Mandrel pin diameter for bending reinforcement

d_b = Reinforcement bar diameter

N_{Rsk} = Number of cycles for resistance of constant amplitude stress $\Delta\sigma_{Rsk}$

NOTES:

- 1 The welding of reinforcement, including non-structural welds (e.g. tack-welds) shall be in accordance with AS/NZS 1554.3.
- 2 The stress range of welded bars $\Delta\sigma_{Rsk}(n_{sc})$ shall not exceed the stress range for straight bar, wire or tendon at the same number of cycles (n_{sc}).
- 3 Valid for all ratios (d_i/d_b) and all diameters d_b .
- 4 The values given in the table are characteristic and do not include the strength reduction factor $\phi_{s,fat}$. These values or higher values shall be validated by testing.
- 5 The values given in the table do not cover coiled and re-straightened bars.

For multiple cycles with variable amplitudes the damage may be added by using—

$$D_{Ed} = \sum_i \frac{n_{sc}(\Delta\sigma_{si})}{N_{Rsk}(\Delta\sigma_{si}/\phi_{s,fat})} \quad \dots 18.8(2)$$

where

$n_{sc}(\Delta\sigma_{si})$ = applied number of cycles for a stress range $\Delta\sigma_{si}$

$N_{Rsk}(\Delta\sigma_{si}/\phi_{s,fat})$ = resisting number of cycles for a stress range $\Delta\sigma_{si}/\phi_{s,fat}$

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For prestressing or reinforcing steel exposed to fatigue loads, the calculated stresses shall not exceed the design yield strength of the steel.

In areas of high fluctuating stresses, welded lap splices shall not be used.

18.9 CALCULATION OF STRESSES IN REINFORCEMENT AND TENDONS OF FLEXURAL MEMBERS

In flexural members fatigue resistance of the longitudinal reinforcement and tendons and the shear reinforcement shall be determined.

The angle between the compression struts and the longitudinal axis of the member shall be chosen to be between 35° and 55° , except that for non-prestressed slabs and trough girders the angle shall be between 40° and 55° .

SECTION 19 JOINTS, EMBEDDED ITEMS AND FIXINGS

19.1 JOINTS

19.1.1 General

A joint in a structure or member shall be designed and constructed so the load-carrying capacity and serviceability of the structure or member is maintained while serving its designed function. Joints shall be for construction purposes ('construction joint') or to control movement ('movement joint').

19.1.2 Construction joints

19.1.2.1 General

Construction joints shall be designed and installed to satisfy intended construction practice for the specific application. Construction joints shall be designed to produce a well-bonded interface between hardened concrete and freshly placed concrete.

19.1.2.2 Joint spacing

Construction joints shall be located to facilitate the placement of concrete in accordance with Clause 17.1.3 and to meet concrete placement restrictions and finishing requirements. They shall be located in regions of minimal shear force and, where possible, in unobtrusive locations. The spacing shall be determined by the rate of concrete placement and finishing on site or as a result of any unplanned interruption to placement operations.

Where an interruption to the placing of concrete occurs such that the requirements of Clause 17.1.3(c) or Clause 17.1.3(d) or Clause 17.1.3(e) cannot be fulfilled, a construction joint shall be made.

19.1.3 Movement joints

19.1.3.1 General

Movement joints shall be designed and constructed to—

- (a) control movement at a joint;
- (b) control cracking at design locations; or
- (c) provide articulation and separation between concrete members in a structure and meet their designed function without impairing the load-carrying capacity or serviceability of either the structure or member.

Movement joints shall be formed between two concrete members to allow movement to occur, typically as a result of shrinkage, creep, diurnal effects and differential settlement. Generally, they will extend throughout the member depth imparting complete discontinuity between adjoining concrete members. However, joints for shrinkage are not necessarily formed through the depth of the member.

19.1.3.2 Joint spacing

In reinforced concrete members, the spacing of movement joints shall take into account effects such as shrinkage, temperature movement, moisture change, creep and other relevant factors. The level of prestress and subsequent member shortening shall also be considered in prestressed concrete members.

19.1.4 Joint fillers and sealants

Material infill in a joint shall remain in place and deform in response to loading and movement without undergoing any change that will adversely affect the functionality of the joint. Such infills shall consist of either a joint filler or sealant (or both). Fillers may be inserted into a joint in a compressed condition either when the concrete is plastic or is in its hardened state.

A sealant shall be employed to provide weatherproofing, fire resistance, acoustic insulation, chemical resistance, prevention of deleterious material ingress or other function that cannot be imparted by a filler.

Joints shall be sealed or otherwise designed and detailed to prevent the entry of dirt or incompressible material into the joint which would detrimentally affect the joint movement or operation. Where loading across the joint is anticipated, the designer shall ensure the infill materials are of sufficient hardness and the concrete edges adequately protected, to prevent joint edge spalling.

19.2 EMBEDDED ITEMS

19.2.1 General

For the purpose of this clause, embedded items include pipes and conduits with their associated fittings, sleeves, permanent inserts for fixings and other purposes, holding-down bolts and other supports.

Items may be embedded in concrete members provided the required strength and serviceability of the member is satisfied, and the durability requirements of Clause 4.10.3.7 are met.

19.2.2 Pipes

Embedded conduits and pipes shall conform with the relevant Standards.

NOTE: For example,

- (a) for electrical purposes.....AS/NZS 3000; and
- (b) for plumbing purposes AS/NZS 3500.

Pipes designed to contain liquid, gas or vapour under pressure or extremes of temperature may be embedded in structural concrete, provided the maximum pressure to which any piping or fitting is designed to be subjected will not exceed 2000 kPa, and the effect that inclusion of the pipe has on the strength and serviceability behaviour of the member is taken into account.

19.2.3 Spacing

The minimum clear distance between embedded items, and between embedded items and bars (including bundled bars), tendons or ducts, shall be sufficient to ensure the concrete can be placed and compacted to conform with Clause 17.1.3.

19.3 FIXINGS

19.3.1 General

Fixings, including holding-down bolts, inserts and ferrules, shall conform with the following:

- (a) A fixing shall be designed to transmit all forces, acting or likely to act on it.
- (b) Lifting inserts or fixings which are intended for lifting or for the attachment of temporary supports shall comply with the requirements of AS 3850.1.
- (c) Fixings shall be designed to yield before ultimate failure in the event of overload.

- (d) The anchorage of any fixings shall be designed in accordance with Section 13. The design strength of this anchorage shall be taken as ϕ times the ultimate strength, where $\phi = 0.6$. In the case of shallow anchorages, cone-type failure in the concrete surrounding the fixing shall be investigated taking into account edge distance, spacing, the effect of reinforcement, if any, and concrete strength at time of loading.
- (e) In the absence of calculations, the strength of a fixing shall be determined by load testing of a prototype to failure in accordance with Paragraph B4, Appendix B. The design strength of the fixing shall be taken as ϕ times the ultimate strength where the ultimate strength is taken as the average failure load divided by the appropriate factor given in Table B4.3, Appendix B and $\phi = 0.6$.
- A1 | (f) The spacing between, and cover to, fixings shall be in accordance with Clause 19.2.3. The cover for fixings shall be in accordance with Section 4. The cover for fire resistance shall be in accordance with Section 5.

19.3.2 Post-installed and cast-in fastenings

Post-installed and cast-in fastenings using mechanical, chemical or cast in anchor channels, shall be designed in accordance with AS 5216.

NOTE: AS 5216 *Design of post-installed and cast-in fastenings for use in concrete* has further information on the minimum requirements for the design of fastenings used to transmit loads to concrete for safety-critical applications.

19.3.3 Cast-in fixings

Cast-in fixings shall be designed in accordance with AS 3850.1, Appendix B.

SECTION 20 PLAIN CONCRETE PEDESTALS AND FOOTINGS

20.1 GENERAL

The provisions of this section apply to—

- (a) plain concrete pedestals, provided the unsupported height of the member is not greater than three times the least lateral dimension; and
- (b) plain concrete pad footings supported by the ground.

The value of ϕ throughout the section shall be determined from Table 2.2.2.

20.2 DURABILITY

Plain concrete members shall conform with the provisions of Section 4. The cover to any reinforcement shall conform with that determined in accordance with the provisions of Section 4.

20.3 PEDESTALS

Pedestals subject to combined flexural and axial load shall be proportioned so the maximum compressive stress under the design actions does not exceed $\phi 0.4f'_c$ and the maximum tensile stress does not exceed $\phi 0.45\sqrt{f'_c}$.

The minimum eccentricity shall be taken as $0.1a$, where a is the cross-section dimension in the direction being considered.

20.4 FOOTINGS

20.4.1 Dimensions

The minimum nominal depth of a footing shall be 200 mm.

When calculating the strength of a footing, the entire cross-section shall be considered and the depth of the footing shall be assumed to be 50 mm less than the nominal depth.

20.4.2 Strength in bending

The design strength under bending shall be based on a linear stress-strain relationship in both tension and compression.

The design strength in bending shall be taken as ϕM_{uo} , where M_{uo} is calculated using the characteristic flexural tensile strength ($f'_{ct,f}$).

The critical section for bending shall be taken at—

- (a) the face of the column, pedestal or wall for concrete members;
- (b) halfway between the centre and face of the wall for a masonry wall; or
- (c) halfway between the face of the column and the edge of the base plate for a steel column and base plate.

20.4.3 Strength in shear

The design strength of a member in shear shall be determined in accordance with either or both of the following:

- (a) Where the member acts essentially as a one-way member, and a shear failure can occur across the width of the rectangular cross-section (b) of the member, the design strength in shear shall be taken as ϕV_u where—

$$V_u = 0.15bD(f'_c)^{1/3} \quad \dots 20.4.3(1)$$

The critical section for one-way shear shall be taken at $0.5D$ from the face of the support.

- (b) Where a shear failure can occur locally around a support or loaded area, the design strength in shear shall be taken as—

$$\phi V_u / \left[1 + (uM^*) / (8V^*aD) \right] \quad \dots 20.4.3(2)$$

where

$$V_u = 0.1uD(1 + 2/\beta_h)\sqrt{f'_c} \leq 0.2uD\sqrt{f'_c}$$

u = effective length of the shear perimeter [see Figure 9.3(A)]

a = dimension of the critical shear perimeter, which is parallel to the direction of bending being considered [see Figure 9.3(B)]

β_h = ratio given in Clause 9.3.1.4

SECTION 21 SLAB-ON-GROUND FLOORS, PAVEMENTS AND FOOTINGS

21.1 GENERAL

This section sets out additional design considerations for slab-on-ground floors and pavements and footings including plain concrete pavements and steel fibre reinforced concrete meeting the requirements of Clause 16.6, but excluding residential floors and pavements designed to AS 2870.

21.2 DESIGN CONSIDERATIONS

The design of pavements and slabs supported by the ground and any joints therein shall take into account, but not be limited to, the following considerations:

- (a) The determination of design loading.
- (b) Soil-structure interaction.
- (c) The influence of the pavement or slab on the behaviour of the other parts of the structure.
- (d) Effects of traffic on joints.
- (e) Differential movement at joints.
- (f) The limitation of moisture passing through the slab or pavement.
- (g) The effect of water pressure, if any.
- (h) Techniques to control and minimize cracking.
- (i) Techniques to minimize shrinkage warping.
- (j) Techniques to minimize differential temperature effects.

21.3 FOOTINGS

21.3.1 Reinforced footings

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Two-way footings—

- (a) minimum flexural reinforcement shall be designed in accordance with—
 - (i) Section 9; or
 - (ii) Section 16;
- (b) minimum tensile reinforcement ratio shall be given by $0.19(D/d)^2 f'_{ct,f} / f_{sy}$; and
- (c) for fibre reinforced concrete, the minimum fibre dosage shall conform with Clause 16.3.3.8.

21.3.2 Plain concrete footings

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Plain concrete footings shall conform with the requirements of Clause 20.4.

APPENDIX A

REFERENCED DOCUMENTS

(Normative)

AS

1012	Methods of testing concrete
1012.1	Method 1: Sampling of fresh concrete
1012.2	Method 2: Preparation of concrete mixes in the laboratory
1012.3.1	Method 3.1: Determination of properties related to the consistency of concrete—Slump test
1012.4	Method 4: Determination of air content of freshly mixed concrete (series)
1012.8.1	Method 8.1 Method of making and curing concrete—Compression and indirect tensile test specimens
1012.9	Method 9: Determination of the compressive strength of concrete specimens
1012.10	Method 10: Determination of indirect tensile strength of concrete cylinders ('Brazil' or splitting test)
1012.11	Method 11: Determination of the modulus of rupture
1012.12	Method 12: Determination of mass per unit volume of hardened concrete (series)
1012.13	Method 13: Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory
1012.14	Method 14: Method for securing and testing cores from hardened concrete for compressive strength
1012.16	Method 16: Determination of creep of concrete cylinders in compression
1012.17	Method 17: Determination of the static chord modulus of elasticity and Poisson's ratio of concrete specimens
1170	Structural design actions
1170.4	Part 4: Earthquake actions in Australia
1199	Sampling procedures for inspection by attributes (series)
1379	Specification and supply of concrete
1478	Chemical admixtures for concrete, mortar and grout
1478.1	Part 1: Admixtures for concrete
1530	Methods for fire tests on building materials, components and structures
1530.4	Part 4: Fire-resistance test of elements of construction
2758	Aggregates and rock for engineering purposes
2758.1	Part 1: Concrete aggregates
2870	Residential slabs and footings
3610	Formwork for concrete (series)
3799	Liquid membrane-forming curing compounds for concrete
3850	Prefabricated concrete elements
3850.1	Part 1: General requirements
3850.2	Part 2: Building construction
5216	Design of post-installed and cast-in fastenings in concrete

AS/NZS

1170	Structural design actions
1170.0	Part 0: General principles
1170.1	Part 1: Permanent, imposed and other actions

1554	Structural steel welding
1554.3	Part 3: Welding of reinforcing steel
1768	Lightning protection
2425	Bar chairs in reinforced concrete—Product requirements and test methods
4671	Steel reinforcing materials
4672	Steel prestressing materials
4672.1	Part 1: General requirements
4672.2	Part 2: Testing requirements
NZS	
1170	Structural design actions
1170.5	Part 5: Earth actions
3101	Concrete structure standard
3101.1	Part 1: Design of concrete structure
ISO	
12491	Statistical methods for quality control of building materials and components
13270	Steel fibres for concrete—Definitions and specifications
EN	
14651	Test Method for Metallic Fibre Concrete—Measuring the Flexural Tensile Strength (Limit of Proportionality (LOP), Residual)
14721	Test Method for Metallic Fibre Concrete—Measuring the Fibre Content in Fresh and Hardened Concrete
14889	Fibres for concrete—Steel fibres (series)
NCC	National Construction Code
ASTM	
C42	Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
BSI	
6744	Stainless steel bars—Reinforcement of concrete—Requirements and test methods

APPENDIX B

TESTING OF MEMBERS AND STRUCTURES

(Normative)

B1 GENERAL

This appendix applies to the testing of a structure or prototype to check that the strength and serviceability requirements of this Standard are met. Methods for testing hardened concrete in place are also detailed. Testing shall be undertaken by persons competent in, and with expertise for, performing such tests.

B2 TESTING OF MEMBERS

B2.1 Purpose of testing

Structures designed by calculation in accordance with other parts of this Standard are not required to be tested. Tests can be accepted as an alternative to calculation (prototype testing), or may become necessary in special circumstances (proof testing), in order to satisfy the requirements of Clause 2.2 with respect to strength and Clause 2.3 with respect to serviceability.

Where testing is necessary, elements of structures or whole structures shall be either—

- (a) proof-tested in accordance with Paragraph B3, to ascertain the structural characteristics of an existing member or structure; or
- (b) prototype-tested in accordance with Paragraph B4, to ascertain the structural characteristics of a particular class of member, which are nominally identical to the elements tested.

B2.2 Test set-up

All measuring equipment shall be chosen and calibrated to suit the range of measurements anticipated, in order to obtain measurements of the required precision. Care shall be exercised to ensure that no artificial restraints are applied to the test specimen. All necessary precautions shall be taken to ensure that in the event of collapse of any part of a structure being tested, the risk to life is minimized and the collapse will not endanger the safety of the structure being tested (for tests on members) and/or adjacent structures.

B2.3 Test load

The test load shall simulate 100% of the design loads for the limit states for strength and serviceability, as required. The test load shall be applied gradually at a rate as uniform as practicable and without impact. The distribution and duration of forces applied in the test shall be representative of those forces to which the structure is deemed to be subject under the requirements of this Standard.

B2.4 Test deflections

The deflections of each test specimen shall be measured with respect to a fixed datum. Deflections shall, as a minimum requirement, be recorded at the following times:

- (a) Immediately prior to the application of the test load.
- (b) Incrementally during the application of the test load.
- (c) Immediately the full test load has been applied.
- (d) Immediately prior to removing the test load.
- (e) Immediately after the removal of the test load.

B3 PROOF TESTING

B3.1 Test procedures

A proof test shall be conducted as follows:

- (a) Before applying any load, record the original position of the members involved.
- (b) Apply the test load as determined from Paragraph B2.3, for the relevant limit state.
- (c) Maintain the test load for the necessary period as stated in Paragraph B3.2.
- (d) Remove the test load.

B3.2 Criteria for acceptance

Criteria for acceptance shall be as follows:

- (a) *Acceptance for strength* The test structure or member shall be deemed to conform with the requirements for strength if it is able to sustain the strength limit state test load for at least 24 h without incurring any significant damage such as spalling or excessive cracking.
- (b) *Acceptance for deflection* The test structure or member shall be deemed to conform with the requirements for serviceability if it is able to sustain the serviceability test load for a minimum of 24 h without exceeding the design serviceability limits.

Appropriate deflection limits for beams and slabs shall be determined using Clause 2.3.2 and the deflections calculated taking into account long-term and short-term effects, allowing for the age and loading history of the structure.

B3.3 Damage incurred during test

The test specimen shall be regularly inspected, to determine the nature and extent of any damage incurred during the test. The effects of the damage shall be considered and the test disbanded if collapse seems likely. At the completion of the test, repairs to damaged parts shall be carried out.

B3.4 Test reports

A report shall be prepared, which shall contain, in addition to the test load-deflection history and serviceability criteria records, a clear description of the test set-up, including the methods of supporting and loading the members, the method of measuring deflections, crack-widths, and so on, and any other relevant data. The report shall also contain a statement as to whether or not the structure, substructure or members tested satisfy the relevant acceptance criteria in Paragraph B3.2.

B4 PROTOTYPE TESTING

B4.1 Construction of prototypes

Prototypes shall be constructed from materials that conform with this Standard, and manufactured in accordance with the specification for the member.

B4.2 Number of prototypes

The number of prototypes to be tested shall be selected so that statistically reliable estimates of the behaviour of the member, at relevant limit state values, can be determined from the results of the testing. No fewer than two prototypes shall be tested.

B4.3 Test load

The test load for strength shall be applied gradually until the total load on the prototype is equal to the design load for the strength limit state as determined by Section 2, and multiplied by the relevant factor given in Table B4.3. This factor shall be selected with respect to the expected coefficient of variation in the parameters that affect the strength and the sample size selected for the testing program, unless a reliability analysis shows that a different value is appropriate.

The total load for each prototype used to assess serviceability shall be the design load for the serviceability limit state as determined from Section 2 multiplied by a factor of 1.2.

TABLE B4.3
FACTOR TO ALLOW FOR VARIABILITY
IN PRODUCTION OF UNITS

Number of similar units to be tested	Expected coefficient of variation		
	10%	20%	30%
2	1.3	1.7	2.3
3	1.3	1.6	2.1
5	1.2	1.5	1.8
10	1.1	1.3	1.5

NOTE: Intermediate values may be obtained by linear interpolation. The above values are based on a target safety index of 3.0 for a confidence level of 90%.

B4.4 Test procedure

The method of applying the test load to the prototype shall reflect the most adverse conditions expected to occur during construction and the in-service condition.

A prototype test shall be conducted as follows:

- (a) Before applying any load, record the original position of the members in the test specimen.
- (b) Apply the test load for the relevant limit state, as determined from Paragraph B4.3.
- (c) Maintain the test load for the necessary period, as stated in Paragraph B4.5.
- (d) Remove the test load.
- (e) Inspect and record the prototype for damage, spalling, cracking and any other relevant observations.

B4.5 Criteria for acceptance

The units represented by the prototypes shall be deemed to conform with this Standard for serviceability and strength where Item (a) is satisfied and Item (b) or Item (c) is satisfied, as follows:

- (a) *Variability* Production units shall be similar in all respects to the prototypes tested, and variability of production shall be equal to or less than the expected variability determined at prototype.
- (b) *Acceptance for strength* The test prototype shall be deemed to conform with the requirements for strength if it is able to sustain the strength limit state test load for at least 5 min without incurring any significant damage, such as spalling or excessive cracking.

- (c) *Acceptance for serviceability* The test prototype shall be deemed to conform with the requirement for serviceability if it is able to sustain the serviceability test load for a minimum period of 1 h without exceeding the serviceability limits appropriate to the member. Deflection limits shall be determined using Clause 2.3.2, taking into account only short-term effects.

Qualitative indicators for the parameters affecting strength and serviceability shall be determined for the expected variability during production. These indicators shall be routinely monitored and measured in manufactured units and used to ensure the actual coefficient of variation in production does not exceed the expected coefficient of variation. Alternatively, manufactured units shall be routinely tested to failure, to determine the coefficient of variation.

B4.6 Test reports

A report shall be prepared in accordance with Paragraph B3.4, except that instead of the requirement in the final sentence of Paragraph B3.4, the report shall contain a statement as to whether or not the prototypes tested satisfied the relevant acceptance criteria in Paragraph B4.5.

B5 QUALITY CONTROL

B5.1 General

This paragraph applies to the assessment of a group of units that are part of a production run of similar units. Paragraphs B5.2, B5.3 and B5.4 identify three methods to routinely assess production. One of these methods shall be nominated by the manufacturer as the means of demonstrating that the manufactured group is similar to the tested prototypes. The routine examination nominated shall include the determination of the variability in a production run by relating key indicators in the sample to the previously performed prototype testing and the application of a test load to each sample.

B5.2 Statistical sampling

A sampling plan, in accordance with AS 1199, shall be established for the routine inspection and testing of a produced batch. Sampling shall be undertaken in accordance with this plan and the selected specimens shall be routinely tested to ensure conformance with this appendix is maintained.

For concrete specified by strength, the methods of production and assessment, taken together, shall provide a reliable operating characteristic curve so that—

- (a) concrete with a proportion defective of 0.05 has a probability of acceptance of not less than 50%; and
- (b) concrete with proportion defective of 0.30 has a probability of rejection of not less than 98%.

B5.3 Product certification

Independent assurance of the claim by a manufacturer or contractor of batch consistency shall be permitted, to ascertain whether a production run or application routinely conforms with the requirements of this appendix.

NOTE: The certification should meet the criteria described in HB 18.28 in order that effective quality planning to control production is achieved.

B5.4 Quality system

Confidence in routine assessment of production shall be achieved where the manufacturer or contractor can demonstrate that an audited and registered quality management system conforming with the requirements of the appropriate or stipulated Australian or international Standard for a quality system is in place.

Such a system shall include a quality or inspection plan and test plan, to ensure product conformity.

B6 TESTING OF HARDENED CONCRETE IN PLACE

B6.1 Application

This paragraph applies to the assessment of the strength and other properties of hardened concrete in place by non-destructive testing, by testing of samples cut from representative test panels, or samples cut from members.

B6.2 Preparation of samples

The samples to be tested shall be representative of the concrete under investigation. Prior to testing, surfaces shall be cleaned to remove oil, laitance, curing compounds and surface treatments.

Where required, test panels shall be made of concrete that is identical in composition and which is placed, compacted and cured in a manner similar to concrete used in the member. Dimensions of test panels shall be such that at least three representative samples can be cut from each panel. Test samples of standard dimensions shall be obtained from the test panels by coring or sawing.

B6.3 Non-destructive testing

Non-destructive testing and assessment shall be specified and carried out in accordance with internationally recognized procedures.

NOTES:

- 1 Non-destructive testing (including impact or rebound hammer, ultrasonic pulse velocity, pullout and abrasion testing, or a combination of techniques) may be used to compare the properties of concrete under investigation with that of a representative sample of known quality. In particular, comparable concrete should be of similar maturity, curing history and mix composition. Alternatively, where specified, values obtained by non-destructive tests may be used directly to assess some properties of concrete.
- 2 Combined non-destructive techniques have been found to substantially improve the order of accuracy of the estimated values compared with those obtained from testing by a single method

B6.4 Tests on samples taken from the structure

B6.4.1 Test requirements

Taking and testing of cores and beams from members and sample panels shall conform with the following:

- (a) Core and beam locations shall be selected so as to minimize any consequent reduction of strength of the structure.
- (b) The cores and beams shall be representative of the whole of the concrete concerned and in no case shall less than three samples be tested.
- (c) Cores and beams shall be examined visually before and after testing, to assess the proportion and nature of any voids, cracks and inclusions present. These factors shall be considered in the interpretation of the test results.
- (d) Cores shall be taken and tested for compressive strength in accordance with AS 1012.14 and beams shall be taken in accordance with ASTM C42. The beams shall be tested for flexural strength in accordance with AS 1012.11, and shall be tested dry unless the concrete concerned will be more than superficially wet in service. The density of cores and beams shall be determined in accordance with AS 1012.12, in the same condition as applicable to testing for compressive strength using AS 1012.1 or AS 1012.2 by sealing or wrapping samples where appropriate.

B6.4.2 *Interpretation of results*

The strength of the concrete in the member shall be estimated as either—

- (a) 1.15 times the average strength of the cores and beams; or
- (b) by using test data from cores or beams taken from another member for which the strength of the concrete is known.

APPENDIX C

RESIDUAL TENSILE STRENGTH TEST FOR SFRC

(Normative)

C1 GENERAL

This section shall apply where direct tension testing is used to determine the residual tensile strength of strain softening steel fibre reinforced concrete.

C2 PROCEDURE AND TESTING

The residual tensile strength shall be obtained using the testing arrangement shown in Figure C1 and shall conform with the following criteria:

- (a) The critical section shall be where the cross-sectional area is a minimum.
- (b) The SFRC mix shall be batched to ensure a uniform distribution of fibres, and shall be placed in the moulds in a manner that does not interfere with the distribution of the fibres. The SFRC shall be compacted using lightly applied external vibration.
- (c) Specimens shall be cured for a period of 28 days by following the procedure in Clause 9 of AS 1012.8.1:2014.
- (d) The specimen shall be connected to the testing machine in such a manner that the machine does not apply a load to the specimen during the process of tightening of the grips and prior to testing.
- (e) One end of the specimen shall be connected to the testing machine through a universal joint such that no moment is applied to the end of the specimen.
- (f) Specimens shall be tested under displacement or strain control such that before cracking the strain over the gauged region does not exceed 25 micro strain per minute until the formation of the dominant crack. After cracking is visible to the naked eye across the full section width, the rate may be increased but shall not exceed 800 micro strain per minute until a COD of 2 mm is attained. After COD of 2.0 mm the rate may be further increased but not exceed 4000 micro strain per minute until test completion.
- (g) Displacement measurements shall be taken on each of the four sides with the COD taken as the average of these measurements.
- (h) A minimum of six specimens shall be tested.
- (i) Tests where the failure of the specimen is outside of the testing region, or where the results are influenced by the test specimen boundaries, shall be retested.
- (j) The characteristic values of the tensile strength $f'_{0.5}$ and $f'_{1.5}$, corresponding to CODs of 0.5 mm and 1.5 mm, respectively, shall be determined statistically as the 95 percentile confidence value assuming the population is normally distributed.
- (k) The mean values of $f_{0.5m}$ and $f_{1.5m}$, corresponding to CODs of 0.5 mm and 1.5 mm, respectively, shall be determined statistically as the 50th percentile confidence value assuming the population is normally distributed.

The stress results obtained from the test shall be multiplied by the three-dimensional orientation factor k_{3Dt} , where—

$$k_{3Dt} = \frac{1}{0.94 + 0.6l_f / b} \leq 1 \quad \dots \text{C2}$$

and l_f is the length of the steel fibre and b is taken as the average of the width and depth of the specimen taken at the critical section.

NOTES:

- 1 The factor k_{3Dt} removes the influence of the boundaries on the fibre distribution and converts the results of the test to a state where the fibres can be considered to be randomly orientated in three-dimensional space.
- 2 Testing should be undertaken in a laboratory accredited by the National Association of Testing Laboratories (NATA).

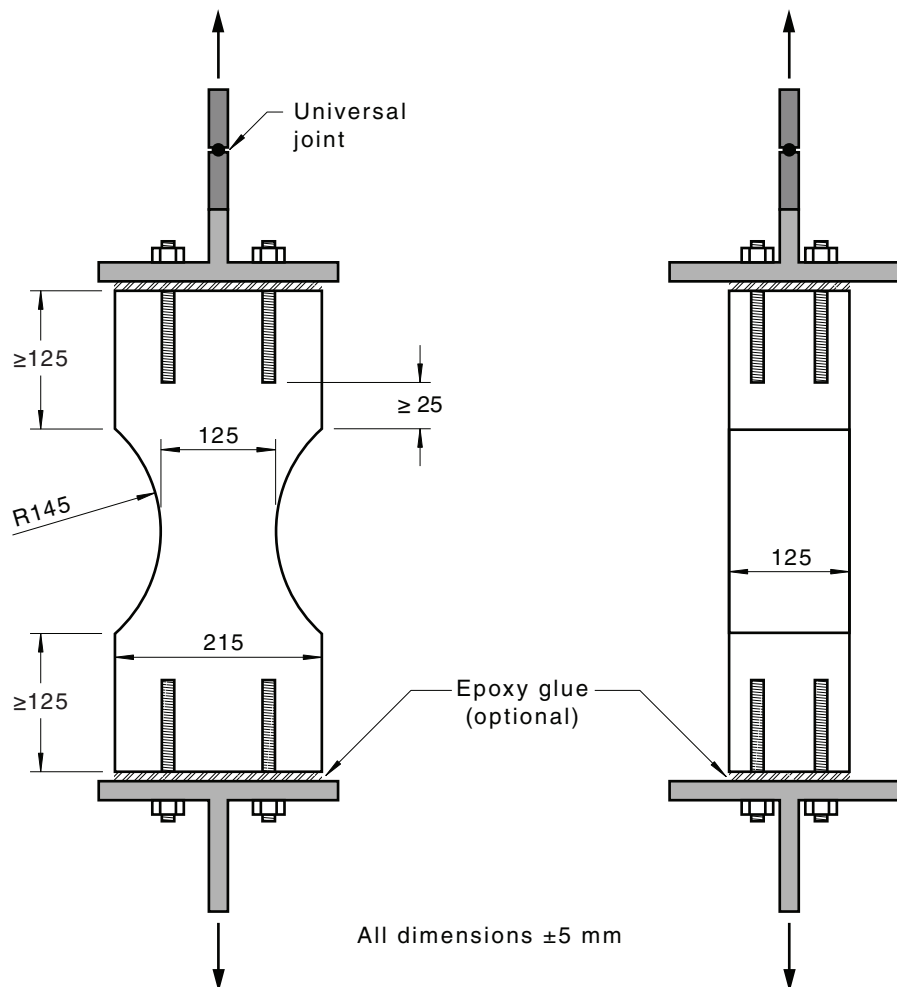


FIGURE C1 TESTING ARRANGEMENT FOR DIRECT TENSION

C3 RECORDS

The following information shall be recorded:

- (a) Identification of sample.
- (b) Identification of specimens.
- (c) Maximum aggregate particle size.
- (d) Type and dosage of fibres.
- (e) Water to cementitious material ratio.
- (f) Course and fine aggregate type and source.

- (g) Specimen dimensions.
- (h) Date and time of moulding.
- (i) Curing history of specimen.
- (j) Compressive strength at the age of testing.
- (k) Load and displacement readings.

NOTE: Where available, the following additional information may be recorded:

- (a) Mix description (specified slump, specified compressive strength or nominal mix proportions).
- (b) Cement brand and type.
- (c) Admixtures – type, brand and quantity.

C4 REPORT

In the event of a report being prepared, the following information shall be included:

- (a) Identification of sample.
- (b) Identification of specimens.
- (c) Maximum aggregate particle size.
- (d) Type and dosage of fibres.
- (e) Water to cementitious material ratio.
- (f) Course and fine aggregate type and source.
- (g) Specimen dimensions.
- (h) Date and time of moulding.
- (i) Compressive strength at the age of testing.
- (j) Load versus crack opening displacement.
- (k) Stress at average crack opening displacement of 0.5 mm.
- (l) Stress at average crack opening displacement of 1.5 mm.
- (m) Maximum stress before cracking.
- (n) Maximum stress after cracking and corresponding average crack opening displacement.
- (o) Such other information in the records as may be requested.
- (p) Any variations from standard conditions.

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 SA
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 CCAA
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AMENDMENT CONTROL SHEET

AS 3600:2018

Amendment No. 1 (2018)

CORRECTION

SUMMARY: This Amendment applies to Clauses 3.1.7.2, 4.10.3.7, 5.6.3, 5.6.4, 8.1.3, 8.1.5, 8.2.3.4, 8.2.4.2.1, 8.2.7, 8.2.8.3, 9.4.4.1, 14.5.2.2, 14.6.7, 17.4.1, 17.4.2, 18.3, 18.4, 19.3.1, 20.4.3 and 21.3.2, and Tables 2.4 and 5.7.2.

Published on 21 November 2018.

Amendment No. 2 (2021)

REVISED TEXT

SUMMARY: This Amendment applies to the Table of contents, global change, Clauses 1.1.2, 1.7, 2.1.2, 2.5.2.2, 2.5.2.3, 3.1.1.1, 3.3.4.2, 4.10.2, 4.10.3.2, 5.3.1, 5.6.3, 5.7.2, 5.7.4.1, 6.2.7.1, 8.1.4, 8.1.5, 8.2.1.2, 8.2.1.3, 8.2.1.5, 8.2.1.6, 8.2.1.7, 8.2.3.1, 8.2.3.2, 8.2.3.3, 8.2.3.4, 8.2.4.1, 8.2.4.2.1, 8.2.4.2.2, 8.2.4.2.3, 8.2.4.3, 8.2.5, 8.2.5.4, 8.2.5.5, 8.2.5.6, 8.2.7, 8.2.8.1, 8.2.8.2, 8.3.1.4, 8.3.2.2, 8.3.3, 8.4.3, 8.6.1, 8.6.2.2, 8.6.2.3, 9.5.1, 9.5.2.1, 9.5.3.2, 10.7.3.1, 10.7.4.3, 11.2.1, 11.5.2, 11.6, 11.6.1, 11.6.3, 11.7.4, 11.7.5, 12.5, 12.5.1, 12.5.2, 12.5.3, 12.5.4, 12.5.5, 12.6, 13.1.2.2, 13.1.2.3, 13.1.2.7, 13.3.2.4, 13.3.3, 14.2.10, 14.5.4, 14.6.7, 16.1, 16.3.3.4, 16.4.2, 16.4.4.2.1, 16.4.7.2.1, 17.6.2.5, 17.7.2, 18.4, 18.8, 21.3.1 and 21.3.2, Equations 8.2.1.5, 8.2.4.2.2(1), 8.2.4.2.2(2), 8.2.8.2(2), 8.4.3, 14.6.6, 10.8(1), 10.8(2), 18.3(1), 18.4(2) and 20.4.3(2), Tables 2.2.2, 2.2.5, 2.5.2.3(B), 3.1.1.1, 3.1.2, 3.2.1, 5.6.3, 5.6.4, 5.7.2, 8.6.3 and 13.1.2.3, and Figures 8.2.8 and 18.8.

Published on 21 May 2021.

NOTES

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